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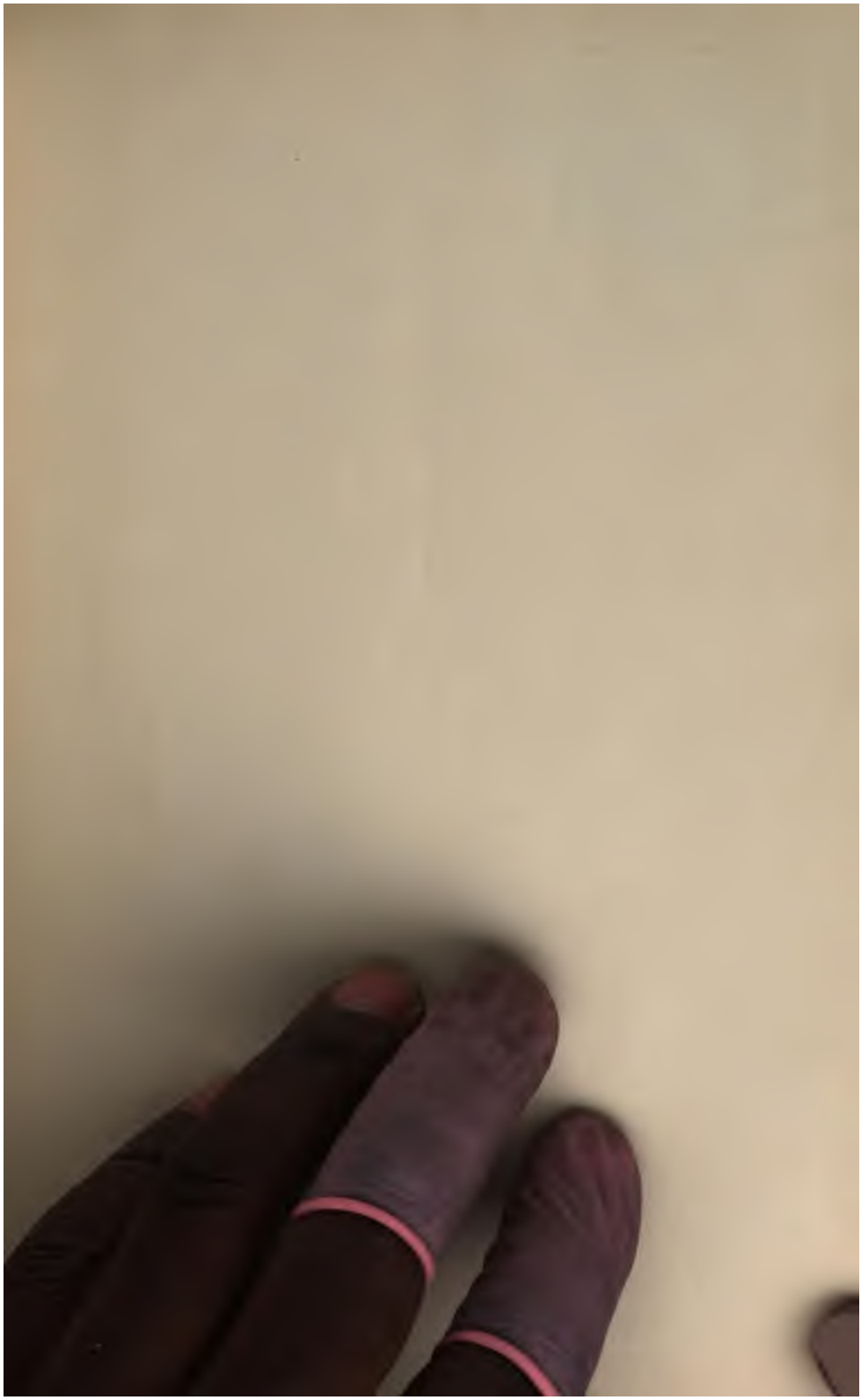

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Radford's Cyclopedia of Construction

Carpentry, Building *and* Architecture

A General Reference Work on

MODERN BUILDING MATERIALS AND METHODS AND THEIR PRACTICAL APPLICATION TO ALL FORMS OF CONSTRUCTION IN WOOD, STONE, BRICK, STEEL, AND CONCRETE; INCLUDING ALSO SUCH ALLIED BRANCHES OF THE STRUCTURAL FIELD AS HEATING AND VENTILATING, PLUMBING, ELECTRIC WIRING, PAINTING, CONTRACTS, SPECIFICATIONS, ESTIMATING, STRUCTURAL DRAFTING, ETC.

Based on the Practical Experience of a
LARGE STAFF OF EXPERTS IN ACTUAL CONSTRUCTION WORK

Illustrated

TWELVE VOLUMES

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Radford's Cyclopedia of Construction

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BEING A SELECTED LIST OF STANDARD WORKS COVERING VARIOUS PORTIONS OF THE GENERAL FIELD OF CONSTRUCTION

In addition to the vast store of up-to-date practical information never before published, the editors have embodied in the Cyclopedia of Construction the cream of the world's best literature in this field, giving in condensed form the substance of everything essential in the experience of the past to enable the worker of the present to meet every problem likely to be met with under the conditions of ordinary practice.

For the sake, however, of those who may be interested in pursuing further study along various special lines, we furnish a list of important works that have appeared covering different portions of the field.

In this connection, the editors wish to express their thanks to American manufacturers and dealers in the machinery and materials of construction, as well as to proprietors and patentees of various special mechanical devices and processes, for valuable data, illustrations and suggestions.

Authorities on Construction

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Preface

THE Building Industry, in its various branches, is more closely identified than any other with the marvelous engineering progress of the present day and the untold possibilities of the future. To put all classes of workers in familiar touch with modern methods of construction and the latest advances in this great field, and to bring to them in a form easily available for practical use the best fruits of the highest technical training and achievement, is the service which the CYCLOPEDIA OF CONSTRUCTION aims to render.

The work is pre-eminently a product of practical experience, designed for practical workers. It is based on the idea that even in the larger problems of engineering construction, it is not now necessary for the ordinary worker in concrete, or steel, or any other form of material, to attempt the impracticable task of exploring all the highways and byways where the trained engineer or technical expert finds himself at home. The theories have been worked out; the tests and calculations have been made; observations have been recorded in thousands of instances of actual construction; and the results thus accumulated form a vast treasure of labor-saving information which is now available in the shape of practical working rules, tables, instructions, etc., covering every phase of every construction problem likely to be met with in ordinary experience. This is perhaps most apparent in the sections on *Cement and Concrete Construction, Plain and Reinforced.* To this subject, on account of its supreme importance as a structural factor of the present day, three entire volumes are devoted, embodying the cream of all the valuable information which engineers have gathered up to date. Much of this practical information now presented in this Cyclopedia, has never before been published in any form. By its use,

anyone is enabled to take advantage of the vast labors of others, and to bring to bear on any problem confronting him the results of the widest experience and the highest skill.

The keynote of the Cyclopedia is found in the emphasis constantly laid on the *practical* as distinguished from the *theoretical* form of treatment, in its total avoidance of the complicated formulas of higher mathematics, and in its reduction of all technical subjects to terms of the simplest and clearest English. Throughout the pages devoted to Steel Construction, for example, the mathematics of the subject have been eliminated to such an extent that *the reader will not find a single instance where even a square root sign has been used.*

In addition to the larger problems of engineering and building construction, one entire volume, as well as many chapters scattered through the work, is devoted to those smaller constructions that are of special interest to the teacher or student of manual training or the home shop worker of a mechanical turn of mind.

Inasmuch as a wider knowledge and a more intelligent grasp of the fundamental principles of construction and design will tend to greater efficiency on the part of workingmen, and to greater economy in production, the purpose of the CYCLOPEDIA OF CONSTRUCTION is one which will appeal strongly, not only to the men themselves, but also to the architectural and engineering fraternity as a whole.

The authors of the various sections are all men of wide experience whose recognized standing is a guarantee of reliability and practical thoroughness.

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GATEWAY TO LODGE ON AN ESTATE NEAR PHILADELPHIA, PA.

Masonry Construction

Stone Masonry

Rocks in Their Natural State. Rocks are divided into three general classes, determined by the manner of their formation in nature's great laboratory. These classes are: (1) **Igneous,** (2) **Sedimentary** or **Aqueous,** and (3) **Metamorphic.**

(1) **Igneous rock** is that which, in some remote geological era, has been thrown up from the earth's interior in a molten state as volcanic lava. Such rock varies greatly in texture and appearance, from the well-known pumice-stone, which is porous and light in weight, to the dense and crystalline granite, so valuable as building material. Some of this lava has been left at or near the surface of the ground, and so has cooled quickly; while some has been covered to a great depth and has remained warm for centuries. The latter condition tends to the formation of large crystals and solid masses, while the former results in fine-grained and glassy stones.

Igneous rock is otherwise divided according to chemical composition, into **acid** and **basic** varieties. In the acid stone, of which granite

may be taken as a type, silicon is always present in large proportion. In the basic variety, of which trap is typical, potassium, magnesium, and iron are predominating features.

(2) All exposed rock is subject to slow disintegration, due to many causes. The action of water and of air are potent in causing rocks to crumble; alternate heat and cold and impact from particles blown about by the wind, produce similar results. The fine pieces thus formed may be deposited, by the action of water, in layers. The addition of pressure for great periods of time transforms this debris into new rock, which is called **aqueous** or **sedimentary.** The most notable example of rock of such formation is the sandstone in such common use.

Most of the limestones have been formed in water by animal life of a very low order. These structures are either in the form of coral reefs or masses of shells, both composed of carbonate of lime extracted from the water of the ocean.

(3) Rock that lies deep in the earth's crust may be subjected to enormous pressure, which, in some cases, is sufficient to alter materially its structure and appearance without changing its composition. Thus the beautiful marbles are formed from ordinary limestone; deposits of peat may become anthracite; and deposits of clay may be changed into slate. This change of character gives rise to the name **metamorphic.**

Characteristics of Building Stones. Any variety of stone is valuable according as its char-

acter suits the requirements of the occasion; and the following brief statements regarding the qualities of some of the more common stones should prove of value in choosing building material. In a general way, the requirements of a good building stone are: (1) **Durability**; (2) **Strength**; (3) **Beauty**; and (4) **Cheapness.** Sometimes more than one of these requirements are necessary, and all are desirable. Usually a compromise is effected between the best, regardless of cost, and the cheapest, if nothing else be considered.

Durability. Many agencies are constantly active in destroying the integrity of building stone. The air in the vicinity of large cities and manufacturing plants is laden with fumes, both acid and alkaline, which attack stone of all kinds. Stones which contain large proportions of lime are especially susceptible to the sulphuric and carbonic acid of smoke. Ammonia and nitric acid of rain water have marked effects upon the surface of exposed stone. All stone absorbs more or less moisture, which changes volume with each change of temperature and so sets up strains between the particles. When the range of temperature is from above to below freezing, the change of volume of the moisture is very marked, and the resulting strains are correspondingly large.

Other destructive or disintegrating agents are fire, friction, and pressure. Fire is especially damaging to granite, which is otherwise, per-

haps, the most desirable stone. Friction is serious on stairways and on door-sills. The pressure of superimposed loads is seldom sufficient to crush stone unless it be applied so as to cause bending. This result is usually unavoidable; and many vertical cracks are seen, even in expensive buildings.

All these destroying agencies are resisted in different degrees by the various kinds of building stones. **Granite,** being composed largely of hard quartz, is but slightly acted upon by the acids present in the atmosphere. Its density also prevents the absorption of much water, and so it is not greatly affected by small changes of temperature. On the other hand, intense heat causes granite to disintegrate rapidly. On account of its hardness and strength, this stone may be used in all places where friction and pressure are expected; and by the stonecutter's art it becomes one of the most beautiful stones as well.

A **sandstone** may or may not be durable. This stone yields to destroying agents, principally on account of the disintegration of the material that cements the particles of sand together. Under the best conditions this cementing material is silica, and the sand is quartz; and when this composition is well consolidated, the resulting stone is comparable with granite in ability to resist the action of the air, and is more durable in the case of fire. On the other hand, sandstones are often cemented by material containing sol-

uble matter, such as clay and iron. A notable example of such stone is the Portland brown sandstone used in the construction of many handsome residences in New York. Added to the tendency to disintegrate through chemical action, is the fact that most sandstones are porous, and are consequently much affected by frost.

Limestones embrace widely different varieties—**coral, chalk,** and **marble** marking the extremes. Except as they absorb moisture, they are fairly durable; and other qualities—such as ease of working, and remarkable beauty when polished—make the best of this material very highly prized.

Slate is a clayey stone that is valuable where transverse strength is imperative. Under friction it does not become excessively smooth, and so has been much used for the paving of walks and for steps. To a greater extent, slate is used as a covering for roofs; and as such, its well-known quality of splitting easily along one axis, while retaining abnormal strength transversely, makes it very well adapted for this purpose. Slate absorbs but little moisture, and is not oversensitive to the attacks of the gases in the air.

Strength of Stone. It may be said that stone is never used in direct tension. So the strength depends upon the ability to resist flexure and compression. Stones are subjected to bending stresses when used as lintels over doors and win-

dows, as covers to drains and culverts, as sidewalks, and in a few other instances.

The breaking stresses per square inch in stone beams subjected to bending, crushing, and shearing, are given in Tables II and III. These values should be divided by four or more, in determining the proper sizes to be used in building.

Stone in the mass, as that under foundations, seldom fails by compression under any loads in use in engineering or architectural construction. The values given in tables of compressive strength of stone, are usually derived by breaking cubes measuring two or four inches on a side. When somewhat larger cubes are tested, the breaking load per square inch is greater, and the natural inference is that the unit-strength increases with the cross-section. Nothing, however, is definitely known of the compressive strength of large blocks of stone, as no testing machines are capable of breaking such specimens. When it is remembered that the weight of all the iron of a railroad track from New York to Chicago could be safely borne by a block of granite twenty feet square, it may cause wonder that stone structures ever fail. In nearly every case of such failure, a close examination reveals other than direct compression. Occasionally an eccentric loading may bring an enormous weight upon a small area, as on the sharp edge of a block, and cause failure by compression; but faulty foundation bed, flexure, or disintegration through the action of natural or artificial, chem-

ical or physical agencies, nearly always are responsible for failures that occur in stonework.

Table III gives the strength per square inch of cubes of stone of various kinds. The results are of value, as strength usually indicates density and ability to resist frosts and acids of the air. It is of course understood that the values in such tables are not to take the place of tests made for a particular stone, when it is possible to have this done. As the table covers quite a range of quarries, it gives a good indication of what may be expected of stone of different kinds found anywhere.

The strength of stone masonry differs materially from that of the stone of which it is formed. The strength of the mortar and the thickness of the joints used are large factors in determining what a structure will support. For the reason mentioned in a paragraph above, no tests have been made on full-sized masonry structures; and so the strength of small cubes of stone and that of the mortar are to be taken as indicating the probable relative strength of the built-up masonry rather than the absolute strength.

Beauty and Cost of Stone. Beauty is a relative term; and a comparison of stones in this respect must usually be a matter of personal preference. Fitness in the choice of stone and of its finish, is the main thing to be sought. A tunnel portal of polished marble in the mountains would

be as lacking in beauty as a rough stone interior might be in a city building.

The cost of stone is subject to so many changes and to so many local conditions, that nothing more than a very general statement can be made concerning it.

A large part of the total cost of stone as used in building, is the expense incurred in cutting the stone. Within the past few years, the following prices for sawing stone have been quoted: Sandstone, 8 to 10 cents; limestone, 15 to 18 cents; marble and granite, 25 to 35 cents—all per square foot. It costs nearly $5.00 to quarry a cubic yard of granite, split to three dimensions. If split in random sizes, the cost is perhaps $4.00. Granite broken out by very heavy blasts, to be afterwards used in this form or to be split into regular shapes, is sometimes quarried as cheaply as 50 cents or less per cubic yard. Bedford limestone costs from $1.50 up per cubic foot, ready for the wall.

TESTS OF STONE

Many of the characteristics of stone are apparent to the practiced eye of the workman. Others are easily seen with the aid of a magnifying glass; while some are determined only after thorough tests by approved methods of physical, microscopical, and chemical examination.

Simple Tests. Some of the simple methods of judging of the fitness of stone for a certain pur-

pose will be described first; and the results of some of the more exact tests will then be shown.

When such a course is practicable, a visit should be made to the quarry, where the first examinations should be directed to the surface of the rock that has been exposed longest to the atmosphere. It is a good sign if this surface shows originally sharp edges still in that condition. If the rock at the bottom and near the top of a vertical face presents a like appearance, the inference is that the upper surface has not yielded readily to atmospheric action. The presence of a deep layer of soft, almost disintegrated rock at the top, indicates that the opposite is true. A comparison of a freshly cut block with a similar one that has seasoned for some months, will often show a marked increase in hardness, with the lapse of even this short time, which will be beneficial. This result is occasioned by the fact that the water which the rock has absorbed sometimes holds in solution some of the cementing constituents of the stone, such as silica.

An examination of some actual example of construction work in which the stone from the quarry in question has been utilized as building material, may also show the weathering qualities and other characteristics. Tool marks and chippings should show plainly for many years, and should feel sharp to the touch. Small, well-defined, dark spots appear in stone containing iron pyrites (iron and sulphur), upon long exposure to rain and to the at-

mosphere. These spots are unsightly, and do not show in new stone. Any change of color from that of the stone when newly split should be noted. Sometimes the change is for the better, sometimes the reverse.

In some surfaces there will be noticed distinct seams or veins of different color and texture. The small magnifying glass and the knife blade will aid in determining whether these irregularities are harder or softer than the other parts of the stone. If the appearance is of little relative importance, these hard seams may not be an objection, while seams of disintegration would be. Again, small cavities, of, say, an inch or two in longest dimension, are blemishes; but if the glass and blade show that they are surrounded by a hard, flinty substance, the stone is not of necessity made materially weaker by their presence. If, however, such cavities show the existence of a seam of softer material, it is a more serious matter, and the specifications will determine whether or not the stone is still acceptable.

If a thin slab of stone be struck with a hammer, it should produce a clear, ringing sound, the resonance being closely proportional to its elasticity.

The color of stone is mainly dependent upon the amount and condition of the iron it contains. The bluish tints indicate that the oxidation of the iron is incomplete, and some further change may be expected before the red or rusty color is

shown. Hence, in general, the reddish stones are the most durable.

Chemical Tests. Such tests can be made only by an expert in a well-equipped laboratory, and an interpretation of the results is important. The value of a stone as a building material depends fully as much upon the physical characteristics as upon the composition. Thus, granite and gneiss are identical chemically; but the latter stone is in layers which cause it to be split readily in one direction and with difficulty otherwise. This characteristic is of course unfavorable if the stone is to be worked, but gneiss is considered to be as durable as any stone. Comparing the granites and sandstones in Table I, it is seen that either may have the larger percentage of quartz, iron, alumina, lime, magnesia, or potash. Carbonic acid is usually present in sandstone in considerable quantities, and is frequently found in minute cavities of the quartz of granite. For the same general locality, a sandstone containing 2 per cent of alumina is apt to be stronger than one having 10 per cent or more, and as a rule the strongest sandstones have a low percentage of this material. The specific gravity and the ability to absorb water furnish no certain criterion of the strength of stones, although, in general, the stronger are the heavier and the less absorptive. A sandstone should not contain so much lime as to cause violent effervescence upon contact with muriatic acid.

TABLE I

Chemical Analysis of Building Stones

Name of Stone	Quartz SiO_2	Iron Oxide Fe_2O_3	Clay (Alumina) Al_2O_3	Lime CaO	Magnesia MgO	Soda Na_2O	Potash K_2O	Carbonic Acid CO_2
GRANITE—	%	%	%	%	%	%	%	%
Chesterfield Co., Va....	80.00	2.77	15.50	1.02	0.15			
Korah Station, Va.....	71.98	1.34	16.63	2.15	0.10	3.15	4.62	
Exeter, Tulare Co., Cal.	75.35	3.94	13.69	2.97	0.06	1.14	2.85	
Rockville, Stearns Co., Minn.....	72.55	2.20	15.92	2.67	0.13	2.50	4.03	
Sioux Falls, Minn., Red	70.10	3.15	16.93	1.63	0.09	0.76	7.32	
JASPER—								
Sioux Falls, Minn.....	98.41	1.42		0.17				
MARBLES—								
Rutland, Vt., White....	1.78	0.86		53.05	1.95			41.79
" " Blue	0.63	0.22		55.26	0.18			43.65
Sutherland Falls, Vt...	1.63	0.25		54.41	0.62			43.00
Fosiliferous, St. Joe, Ark	1.42	2.95		53.30	0.20			42.08
Richville, De Kalb, N. Y., White.....	4.04		1.64	33.01	16.46			43.92
LIMESTONES—								
Isle La Motte, Vt.....	1.29	1.12		54.56	0.18			42.87
Wasioja, Dodge Co. Minn	4.97	1.49		8.07	37.60			47.83
Darlington, Rockwood, Ill.....	0.50	1.45		54.20	1.23			42.61
Bedford, Ind., Blue.....	1.15	1.91		53.25	1.24			42.40
" " Buff.....	0.77	0.63	3.00	52.85	1.18			41.54
Junction City, Kan.....	3.06		4.53	51.70	0.56			40.14
Beaver, Carroll Co., Ark	10.46	1.25		27.97	18.33			41.95
Roosevelt Dam, Ariz...	27.10		2.50	21.80	13.39			35.21
SANDSTONES—								
New England Brown, Cromwell, Conn...	62.47	3.65	10.70	10.25	0.53	2.20	4.15	6.00
Jasper, Ala.....	78.58	2.54	15.19		0.65			3.00
Kettle River, Pine Co., Minn.....	97.10		2.20	0.60	0.10			
Cabin Creek, Johnson Co., Ark.....	91.56	1.25	5.88	0.70	0.20			
Bandera, Radfield, Kan	77.32	3.40	9.45	2.00		2.72	0.67	4.30
Piedmont, Alameda, Cal	70.29	3.94	16.61	3.01				3.00
Coquille River, Coos Co., Ore	60.35	6.86	25.00	3.12	2.13			0.90
Tenino, Thurston Co., Wash.....	67.25	4.57	18.81	6.13	0.10			3.04
Dawson, Ind. Ty.....	75.50	2.31	19.59	0.90				
Brown, Hummelstown, Pa.....	88.13	1.77	5.81	0.20	0.53	0.06	2.63	
Potsdam Sandstone....	99.42		0.31					
SLATES—								
Lancaster Co., Pa	60.32	7.83	19.70	1.12	2.20	2.20	3.18	

TABLE II

Transverse Strength and Elasticity of Building Stones

Names of Stones	Transverse Strength	Coefficient of Elasticity
	Pounds per Square Inch	Pounds per Square Inch
GRANITES—Specimens 4 in. x 6 in. x 24 in. long		
Braddock Quarries, Little Rock, Ark.	1,704	7,067,000
White Rock Mountain, Milbridge, Me.	2,069	9,800,000
Rockville, Me.	1,428	9,420,000
Drake Quarry, Sioux Falls, Minn.	1,378	6,010,000
Branford, Conn.	1,415	5,660,000
Troy, N. H.	2,335	4,800,000
MARBLES—		
Rutland, Vt., White	1,202	4,455,000
Rutland, Vt., Dark	2,109	7,267,000
Sutherland Falls, Vt.	3,054	12,580,000
St. Joe, Ark.	1,615	8,210,000
De Kalb, N. Y.	1,144	12,000,000
Kenesaw Quarry, Tate, Ga.	1,553	
Creole Quarry, Tate, Ga.		4,060,000
LIMESTONES—		
Isle La Motte, Vt.	2,493	14,720,000
Mount Vernon, Ky.	1,434	4,000,000
Beaver, Ark.	2,860	6,645,000
Bowling Green, Ky.	1,317	9,290,000
Bedford, Ind.	1,867	7,160,000
SANDSTONES—		
Cromwell, Conn.	2,243	7,711,000
Worcester Quarry, E. Long Meadow, Mass.	987	1,845,000
Kibble Quarry, E. Long Meadow, Mass.	1,273	1,650,000
Cabin Creek, Ark.	2,442	3,900,000
Fort Smith, Ark.	1,761	3,530,000
Olympia, Wash.	2,073	3,277,000
Chuckanut, Wash.	2,010	2,120,000
Tenino, Wash.	667	1,020,000
ROOFING SLATES—Specimens ¼ in. x 16 in. x 24 in. long		* 12,250,000
Green, mean of 6 specimens	8,615	
Red, mean of 6 specimens	8,325	
Purple, mean of 6 specimens	8,737	
FLAGGING—Specimens 1 in. x 12 in. x 54 in. long		
Green, mean of 3 specimens	9,585	
Red, mean of 3 specimens	8,253	* From Monson, Me.
Purple, mean of 3 specimens	7,915	

Physical Tests of Building Stone. The usual physical tests are of **tension, compression, shear, flexure, specific gravity,** and **absorption.** In

TABLE III

Compression and Shearing Strength of Building Stones

Description	Strength Pounds per Square Inch	
	Compression	Shear
GRANITES—		
From Braddock quarries, near Little Rock, Ark.	19,599	2,088
From Milbridge, Me., "White Rock Mountain"	19,917	2,820
From Rockville, Stearns Co., Minn.	18,121	1,948
Drake's Granite, from Sioux Falls, Minn.	18,175	2,086
From Concord, N. H. (3.06 in. x 3.07 in. x 3.08 in.)	30,330	.....
From Concord, N. H. (3.07 in. x 3.06 in. x 3.03 in.)	23,860	.....
MARBLES—		
Rutland, Vt., White	11,892	1,028
Rutland, Vt., Blue	13,864	1,216
Mountain, Dark, from Vermont	12,833	1,453
Sutherland Falls, Vt.	16,156	1,564
From Tacoma, Wash. (2.03 in. x 2.03 in. x 2.04 in.)	8,910	.....
From Tacoma, Wash. (2.03 in. x 2.03 in. x 2.04 in.)	9,240	.....
LIMESTONES—		
From Isle La Motte, Vermont	14,622	2,173
" Beaver, Carroll Co., Ark.	20,581	1,998
" Bowling Green, Ky.	6,042	1,210
" Bedford, Ind., Buff	9,012	1,222
" " " Blue	10,823	1,016
" Roosevelt Dam, Ariz. (2.0 in. cubes)	5,590	.....
" " " " (2.0 in. cubes)	4,970	.....
SANDSTONES—		
From Cromwell, Conn.	16,894	1,525
" Jasper, Ala.	15,680	2,202
" Cabin Creek, Ark.	18,468	2,479
" Quarries near Fort Smith, Ark.	12,765	1,765
" Olympia, Wash.	12,665	1,642
" Chuckanut, Wash.	11,889	2,016
" Dawson, Ind. Ter.	5,590	.....
" " " " 3.8 in. cubes " " " " 3.9 " " "	4,970	.....

Tables II and III will be found the results of recent experiments, made for the most part at the United States Arsenal at Watertown, Mass., and published as public documents entitled "Tests of Metals." Table IV, giving weights and ab-

TABLE IV

Weight per Cubic Foot, and Rate of Absorption, of Building Stones

Source	Description	Weight	Absorption
Penryn, Cal.	Granite	173.45	.032
Rocklin, Cal.	"	167.25	.054
Gunnison, Colo.	"	169.36	.006
Stony Creek, Conn.	"	165.4	.005
Huron Island, Mich.	"	163.7	1/[illegible]
Watab, Minn.	"	168.4	1/[illegible]
Rockport, Mass.	"	163.2	1/[illegible]
Keene, N. H.	"	166.0	1/[illegible]
Madison Avenue, N. Y.	Gneiss	182.5	.....
Bedford, Ind.	Limestone	152.4	1/[illegible]
Bedford, Ind.	"	140.3	1/[illegible]
Red Wing, Minn.	"	162.2	1/[illegible]
Billingsville, Mo	"	145.0	1/[illegible]
Dorset, Vt.	Marble	164.7	1/[illegible]
Portland, Conn.	Sandstone	148.5	1/[illegible]
Gunnison, Colo.	"	128.9	.146
Fort Snelling, Minn.	"	138.8	1/[illegible]
Medina, N. Y.	"	150.6	1/[illegible]
Cleveland, Ohio	"	140.0	1/[illegible]
Berea, Ohio	"	134.0	1/[illegible]

sorption, is from other sources believed to be entirely reliable. Some of the universities maintain extensive testing laboratories, and publish the results of work done, in bulletins, which are for free distribution.

Table III gives the results of tests of the compressive strength of cubical blocks of stone measuring 4 inches on a side unless otherwise stated. The shearing specimens were 4 inches by 6 inches and 12 inches long. The transverse tests in Table II were on specimens 4 inches by 6 inches and 24 inches long unless otherwise stated. The manner of loading for shearing and transverse tests, is shown in Figs. 1 and 2.

It may be explained that the coefficient of elasticity is found by dividing the stress per square inch by the change of length per linear inch.

Besides these tests, the attempt is sometimes made to discover the probable resistance to frosts and to acids of the atmosphere. A rather popular experiment, known as **Brard's test,** is designed to simulate the action of water when

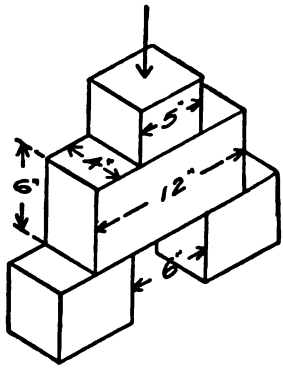


Fig. 1. Manner of Loading for Shearing Test.

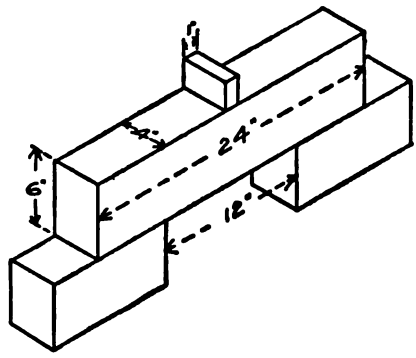


Fig. 2. Manner of Loading for Transverse Test.

freezing. The process is to boil a cube of stone for half an hour in a saturated solution of sulphate of soda. Upon drying, the salts absorbed by the stone crystallize, and, by expansion, produce a rending effect similar to that of frost. After the operation has been repeated for, say, a month, the stone will have suffered a perceptible loss in weight, and, by comparing the losses of several varieties of stone, an inference may be drawn as to the relative weathering qualities.

It is rather unfortunate that the salt has been found to exert a chemical as well as a physical effect, which, to a certain extent, discredits the result. For that reason, this test is not nearly so much used as formerly. If time affords, simi-

RESIDENCE AT LINCOLN, NEB.—CEMENT BLOCKS AND BRICKWORK.

lar results may be attained by alternate drying, soaking, and freezing by artificial means.

The chemical action of the atmosphere is tested by immersing the samples in vessels of water through which carbonic acid is kept constantly passing, and noting the changes of the stone as to color, smoothness and weight.

Stones for stairs, door-sills, and sidewalks are tested as to wearing qualities, by subjecting them to the action of **sand-blasts.** The most suitable stones for such uses should resist wear, and always remain rough. The best test of this kind, however, is afforded by the appearance of the face of the rock in the quarry.

Preservation of Stone. The first rule to be observed is to lay stone on its natural bed if it be of stratified variety. This procedure makes vertical pressures perpendicular to the stratification, and the stone absorbs less water, besides being stronger. Neglect of this precaution has had a marked effect in defacing the brown stone ashlar fronts of many handsome residences.

In the design of outside stonework, the attempt should always be made to leave no depression where water can accumulate; and drainage outward from the wall should be provided for every point on the outside surface. The grooves often cut on the under side of door and window sills are for the purpose of causing the rain and moisture to collect in drops at its edge, rather than to be drawn inward toward the face of the wall.

It is always well to use only stone that has been quarried in the summer and allowed to dry slowly under cover for several months. There is always more or less quarry moisture in newly blasted rock; and by the process of drying, additional cementing material held in solution is deposited in the minute interstices between the grains. For this reason, the drying must for a long time be confined to the thin outside surface of the stone; and experience seems to show that it is unwise to break this skin by dressing or carving.

There is a noticeable difference in the strength and weathering quality of stones dressed in various ways. The method or the tool that finishes the surface with the least impact is the best. A marble or limestone that has been sawn is often, to a marked degree, stronger and less likely to flake off than is the same stone when dressed by heavy hammers. The frequent redressing of surfaces stained by soot is a prolific cause of disintegration.

From the above, it is evident that the **most effective means of delaying disintegration of stone is to prevent the absorption of moisture.** One case of deterioration that caused widespread anxiety in this country, was that of the obelisk known as "Cleopatra's Needle," which was brought from Egypt to Central Park, New York City. This monument had been standing for many centuries, and its hieroglyphic symbols were unimpaired when it was moved to America.

A few years in the damp, gas-laden atmosphere of the American metropolis caused rapid decay, and means of preservation were sought. As described in the Transactions of the New York Academy of Science (November, 1885), the surface was washed with a preparation consisting of paraffine containing creosote, dissolved in turpentine. The mixture was applied, at a temperature of 140 degrees Fahrenheit, to the stone, which had been heated to about the same degree. This treatment is attended by considerable expense, but has been reasonably successful.

Soap and Alum. A cheap and highly effective method of making mortar and masonry impervious to moisture is known as **Sylvester's Method.** This is more often applied to brick than to stone masonry, but may be used in the mortar in either case. It is also used for waterproofing concrete. As described in the Transactions of the American Society of Civil Engineers (Vol. I), the Sylvester process consists in using two washes or solutions for covering the surface of the walls—one composed of Castile soap and water, and one of alum and water. The proportions are three-quarters of a pound of soap to one gallon of water, and half a pound of alum to four gallons of water, both substances being perfectly dissolved in the water before being used. The walls should be perfectly clean and dry, and the temperature of the air not above 50 degrees Fahrenheit when the composition is applied.

The first coat or **soap-wash** should be laid on when boiling hot, with a flat brush, taking care not to create a lather on the brickwork. This wash should remain 24 hours, so as to become dry and hard before the second coat or **alum wash** is applied, which should be put on in the same manner as the first. The temperature of this wash, when applied, may be 60 degrees or 70 degrees Fahrenheit; and this also should remain 24 hours before a second coat of the soap-wash is put on. These coats are to be applied alternately until the walls are made impervious to water. The alum and soap thus combined form an insoluble compound, filling the pores of the masonry, and entirely prevent the water from entering the walls.

While it cannot be said that this method has become very general in use, it is to be recommended. Instead of being applied as a wash, the alum and soap may be mixed with the cement with good results. The present practice is to mix an amount of alum equal to 1 per cent of the weight of the water with the dry cement. Three times this weight of soap is then dissolved in the water before the latter is applied to the cement. It is better to use greater rather than smaller proportions of the alum and soap than those just stated, although the effect of the mixture is to decrease slightly the tensile strength of the mortar.

In case aluminum sulphate is used, one part is mixed with the cement, to four parts of hard

soap, or rather more of soft soap. In using these preventatives as a wash, it is best to follow the original instructions and apply them while hot.

Not a few other washes have been suggested and tried for the purpose of excluding moisture from walls, but the question of their complete success is still debated. Coats of hot asphalt or tar are often applied to underground foundations or walls, with fairly good results. Sometimes tar felt is imbedded in the concrete of walls and foundations for the same purpose.

STONECUTTING

Masonry is classified (1) according to the kind of material of which it is composed—as **Brick Masonry, Stone Masonry,** and **Concrete Masonry;** (2) according to the manner of dressing the faces and edges—as **First-Class** or **Second-Class Masonry, Ashlar, Rubble, Square Stone Masonry, Pitch-Faced** or **Quarry-Faced Masonry;** and (3) according to the arrangement of the material in the wall—as **Uncoursed** or **Dry Rubble, Range** or **Broken Range Work,** and many other similar terms.

In this treatise, only stone and brick masonry will be considered in detail, concrete masonry being reserved for special treatment elsewhere.

Manner of Dressing Stone. Stones used in masonry are divided into three classes—roughly broken stones as they come from the quarry, with little attempt to make them of regular

shape or size; stones roughly broken to conform to some prearranged form and surface; and stones accurately fashioned according to a definite design.

Rough broken stone is used when weight is essential, as in breakwaters, as a pavement for shore protection, as retaining walls where space

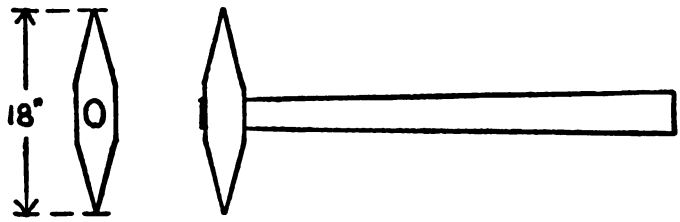


Fig. 3. Pointing Hammer.

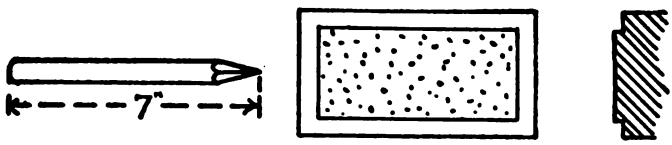


Fig. 4. Hand Pointing Tool and Fine-Pointed Surface.

is not limited and appearance is not a factor, and in foundations for walls. The stone may or may not be laid in mortar. When masonry is divided into numbered classes, such construction would be **third-class**, but it is usually called **rubble masonry**.

Squared stones are broken by hammers or axes to nearly straight edges, and the sides are made approximately plane surfaces. The angles are not always square, and a section may be any polygon with obtuse angles, although this form is exceptional. This class and the class follow-

ing merge into each other almost imperceptibly, the amount of care taken to secure straight edges and plane sides determining the designation. If, however, the only tools used are those mentioned above (hammers and axes), there is little doubt as to the classification. Such work is often called **second-class masonry.**

Cut stones are those dressed to line edges and smooth surfaces. The lines are usually straight, and the surfaces are usually planes, but they

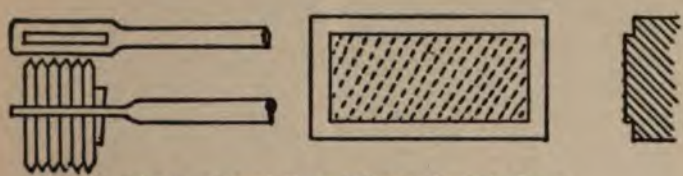


Fig. 5. Crandall, and Crandalled Surface.

may be otherwise. The different varieties of this class are given names corresponding to that of the tool used in shaping them. A few of the common varieties will be described.

Rough-Pointed. A sharp-pointed hammer or a pointed hand-tool used with a hammer or mallet, is employed to bring the face of the stone to such a surface that no point is more than half an inch above or below the surface of the pitch line.

Fine-Pointed. A finer pointing tool applied a greater number of times to the surface, produces a smoother effect which is always the final finish of the stone. The pointing tools and the pointed surfaces are shown in Figs. 3 and 4.

Crandalled. A tool like that shown in Fig. 5 is called a **crandall,** and is designed to do the work of the point more quickly and to produce a more uniform result. The tool is made by wedg-

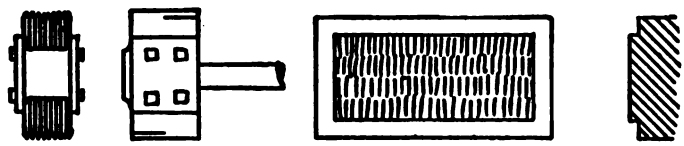


Fig. 6. Patent Hammer, and Axed and Patent-Hammered Surface.

ing several double-pointed bars about 9 inches long into a handle as shown; and it is used with two hands.

Axed and Patent-Hammered. Axes and patent hammers make straight marks instead of conical holes in the stone; and these marks are made parallel to one another over the entire surface to be dressed (Fig. 6). The ax makes one mark, and the patent hammer several marks, at each blow. If the edge of the ax be cut to form teeth, the tool is called a **tooth-ax.**

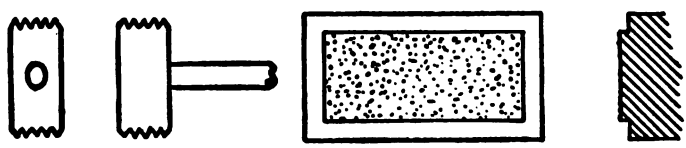


Fig. 7. Bush-Hammer, and Bush-Hammered Surface.

Bush-Hammered. The bush-hammer is a heavy tool, about three inches square in cross-section, with faces cut to form numerous square, pyramidal points. It is used for the final dress-

ing of a surface where the point and ax have been used. This manner of finishing is destructive to the softer stones, causing them to weather badly. (See Fig. 7.)

Many of the soft sandstones and limestones are capable of being worked by machines similar to those used for working wood and iron. Such stone may be **planed, mortised, fluted, grooved,** and **turned** with great facility, and the surface is given a smooth or ribbed finish at will. Cut stone tooled in any of the above-mentioned ways is called **ashlar** or **first-class masonry.**

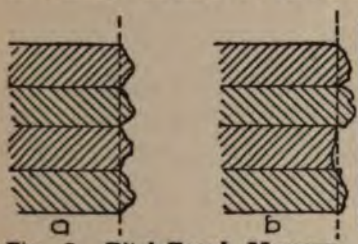


Fig. 8. Pitch-Faced Masonry (a) and Quarry-Faced Masonry (b).

Fig. 9. Range Work.

Arrangement of Stones in the Wall. The arrangement depends partly upon the manner in which the stones have been cut, and largely upon the desired stability and appearance.

Pitch-Faced and Quarry-Faced Masonry. The sketch, Fig. 8, shows clearly the distinction between the two methods of laying the stone. It will be noted in **a,** that both the upper and lower front edges of the **pitch-faced** stone are in the same plane. This arrangement requires considerable more care than is necessary in break-

ing and laying the quarry-faced masonry shown in b.

Range Work. In this construction a course of any thickness, once started, is continued across the entire face; but all courses need not be of the same thickness, as shown in Fig. 9.

Broken Range. When the stone available is

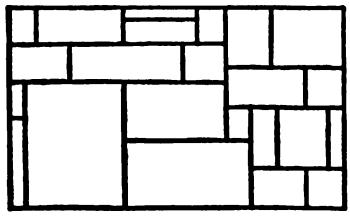


Fig. 10. Broken Range Work.

in rectangular shapes, but quite unequal in size, it is often necessary to break the continuity of the courses at frequent intervals so as to include stone of which there is not enough to make an entire course. This style of construction is shown in Fig. 10.

Uncoursed Rubble. Masonry included under this name is usually made up of but roughly broken stone, or of bowlders of unequal size. This work is usually rough and cheap, but it may also include very firm and expensive construction in which the faces of the stones are left rough but the sides are accurately dressed to thin joints. Fig. 11 shows uncoursed rubble masonry.

Coursed Rubble. Where rubble work is, at frequent intervals, brought to level surfaces, it is said to be **coursed**, as shown in Fig. 12. There

is an analogy between the distinctions of range work and of rubble work; but the latter is made of irregular-shaped stones, while the former is composed of rectangular pieces.

DICTIONARY OF MASONRY TERMS

The following definitions are in common use in the mason's and stonecutter's trade:

Abutment—A supporting wall carrying the end of a bridge or span and sustaining the pressure of the abutting earth.

Fig. 11. Uncoursed Rubble Masonry.

Fig. 12. Coursed Rubble Masonry.

Arris—The external edge formed by two surfaces of stone, whether plane or curved.

Ashlar—A squared or cut block of stone, usually with rectangular dimension.

Backing—The portion of a masonry wall or structure built in the rear of the face. It must be attached to the face by proper bonds, and is usually of a cheaper grade of work than the face.

Batter—The slope or inclination of the face or back of the wall to the vertical.

Bed—The top and bottom of a stone (see also Course-Bed; Natural Bed; Foundation-Bed).

Bed-Joint—A horizontal joint, or one perpendicular to the line of pressure.

Bond—The mechanical distribution of stone, brick, or other building blocks, by overlapping to break joints.

Build—A vertical joint.

Centering or **Centers**—A temporary support of an arch during construction.

Coping—A top course of stone slightly projecting, primarily to protect the masonry from the weather or to better distribute the load, and sometimes added chiefly for ornament.

Course—A separate layer in stone or other masonry.

Course-Bed—Stone, brick, or other building material in position, upon which other material is to be laid.

Cramps—Bars of metal having ends turned at right angles to the bar, to fit into holes in adjacent stones.

Dimension Stone—Blocks of stone cut, or quarried to be cut, to specified dimensions.

Dowels—Straight bars of metal fitted into opposing holes in adjacent faces of stones. The object of a dowel is to prevent movement of stones with reference to each other.

Draft—A narrow strip cut on the face of the stone to the width of the chisel, and usually around the edge, leaving the part inside the draft higher and rougher.

Expansion Joint—A vertical joint or space to allow for temperature changes.

Extrados—The upper curve of a right section of an arch, perpendicular to the axis.

Face—The exposed surface of a stone, in elevation.

Footing—A projecting bottom course.

Foundation—That portion of a structure, usually below the surface of the ground, which distributes the pressure upon its support. Also applied to the natural support itself.

Foundation-Bed—The surface upon which a structure rests.

Grout—A thin mortar either poured or applied with a brush.

Header—A stone which has its greatest length at right angles to the face of the wall, and which bonds the face stones to the backing.

Intrados—The lower curve of a right section of an arch, perpendicular to the axis.

Joint—The narrow space between adjacent stones, bricks, or other building blocks usually filled with mortar.

Lagging—Strips used to carry and distribute the weight of an arch to ribs or centering during construction. Also used to designate the strips temporarily supporting soft material, as outside the timbering of a tunnel.

Lewis—A four-piece steel instrument used in lifting stone. The lewis, when in use, is keyed into a triangular hole cut in the stone.

Lock—Any special device or method of construction used to secure the bonding.

Mortar—A mixture of lime or cement with sand

and water. It is used to cement the various stones in a structure together or to cover the surface of the same.

Natural Bed—The surface of a stone parallel to the stratification.

Parapet—A wall on the edge of an elevated structure, for protection or ornament.

Paving—Regularly placed stones or bricks forming a floor.

Pier—An intermediate support for arches or other spans.

Pitch—To square a stone.

Pitch Stone—Stone having the arris clearly defined by a line beyond which the rock is cut away by the pitching chisel, so as to make approximately true edges.

Pointing—Filling the joints or defects in the face of a masonry structure.

Retaining Wall—A wall for sustaining the pressure of earth or filling deposited behind it.

Ring Stones—The end voussoirs of an arch.

Riprap—Rough stones of various sizes placed irregularly and compactly to prevent scour by water.

Rubble—Field stone or rough stone as it comes from the quarry.

Rubbed—A fine finish made by rubbing with grit or sandstone.

Set—A change from a plastic to a hard state.

Skewbacks—The top course of stones on an abutment. Their upper surfaces are cut at such an angle that the surfaces are approxi-

mately perpendicular to the direction of thrust of the arch.

Slope Wall—A wall to protect the slope of an embankment or cut.

Soffit—The under side of a projection.

Spall—A chip or small piece of stone broken from a large block.

Spandrel Wall—The wall at the end of an arch above the springing line and extrados of an arch, and below the coping or string course.

Springer—The first arch stone above a skewback.

Springing Line—The upper and inner edge of the line of skewbacks on an abutment.

Stretcher—A stone having its greatest length parallel to the face of the wall.

String Course—A course of voussoirs of the same width, perpendicular to the axis of the arch, which extends from one arch face to the other.

Voussoirs—The individual stones forming an arch. They are in the form of truncated wedges.

Wing Wall—An extension of an abutment to sustain the adjacent earth.

INSPECTION OF MASONRY

Familiarity with the following general working rules will enable one readily to distinguish good and bad masonry work:

(1) Beds and joints or builds shall be square with each other, and dressed true and out of

wind. A corner of a stone is strong according as its angle is obtuse; in a wall, this occasions an acute angle adjacent to it, and so it is better that all stones be square.

(2) All stones shall be dressed for laying on natural beds. For granite and marble, this precaution is usually unimportant. With sandstone it must always be observed.

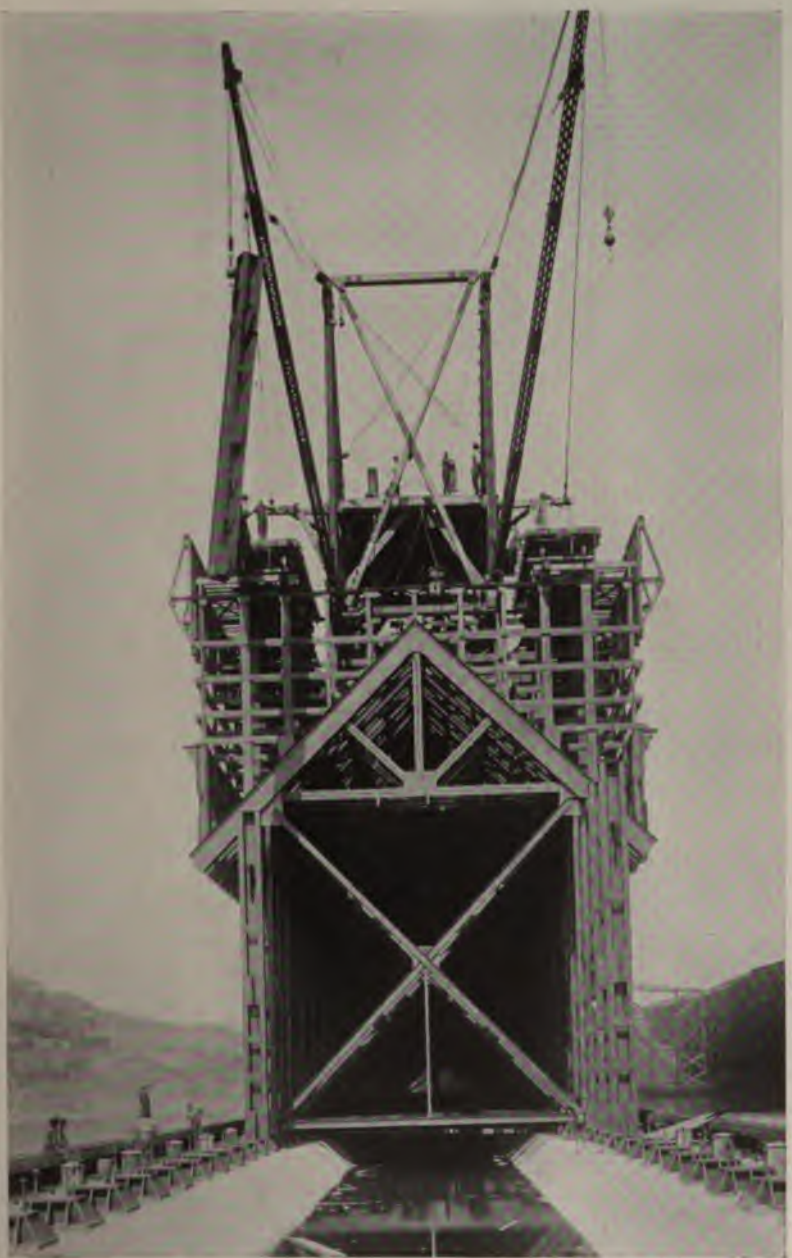
(3) The sand and cement should preferably be mixed dry (in no case should the cement be wet before the sand); then water added, and the whole remixed until the entire mass of mortar is thoroughly homogeneous and leaves the hoe or shovel clean when the latter is drawn from it.

(4) All stones shall be laid on natural beds. Each stone shall be settled into place in a full bed of mortar. That is to say, no stone shall be allowed to touch another stone, but must have a complete coating of mortar around it. If this precaution be not observed, the imposed weight will cause unequal pressure on the stones touching each other; and rupture may occur from this cause.

Such defective work in placing stone can easily be detected by moving it slightly with the foot; if the stone touches another, the jar is distinctly felt. This specification refers principally to rubble work.

(5) No stone shall be dropped or slid over the wall, but shall be placed without jarring the stones already laid. After the mortar has par-

PORCH BUILT OF CONCRETE BLOCKS—RESIDENCE, OF BRICK.



Courtesy of American Bridge Co.

ORE DOCK UNDER CONSTRUCTION AT DULUTH, MINN.

Weight of steel work, 10,000,000 pounds.

PLATE 1—STEEL CONSTRUCTION.

tially set, a jar may destroy the adhesion to the stone, and further setting will not entirely repair the damage.

(6) No heavy hammering shall be allowed on the wall after a course has been laid. It is a custom with careless masons to get a stone in place, and then, with a heavy hammer, break off any projection beyond the face of the wall. This is objectionable for the reason given in (5).

(7) If a stone becomes loose after the mortar has set, it shall be relaid with fresh mortar.

(8) Each stone shall be cleaned and dampened before laying. The stone is moistened so that it shall not draw the water too rapidly from the mortar, and it is cleaned to promote adhesion of the mortar.

(9) Stones shall not be laid in freezing weather, unless by special permission. If this is allowed, the stones shall be freed from ice, snow, or frost by warming, and shall be laid in mortar made of heated sand and water, or, with proper precautions, mixed with brine in the proportion of one pound of salt to 18 gallons of water, when the temperature is at the freezing point. For each degree of temperature below freezing, an additional ounce of salt is used.

(10) Mortar to the depth of one inch shall be removed, before it has set, from the joints and beds from the exposed face of the wall. After the mortar in the wall has completely set, and the stone is free from frost, the joints shall be wet and filled with mortar made of one part

Portland cement and three parts sand. This mortar shall be pounded in with a calking tool, and finished with a beading tool used with a straight-edge.

(11) First-class masonry shall consist of headers and stretchers alternating; at least one-fourth of the wall shall consist of headers extending entirely through the wall. Every header shall be immediately over a stretcher of the un-

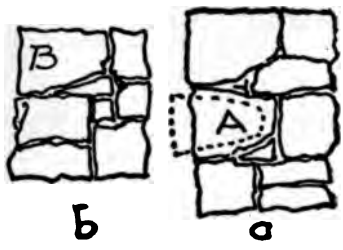


Fig. 13. Use of Blind Headers.

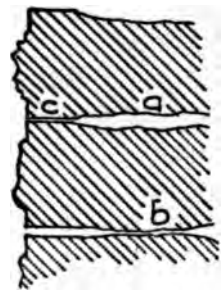


Fig. 14. Flushed Joint.

derlying course. The stones of each course shall be so arranged as to form a proper bond with the stones of the underlying course. A bond of at least one foot will be required. In general, the stones should overlap the stones below an amount at least equal to the depth of the stone.

Blind headers are stones of the proper dimensions on the face, but extending only a short distance into the wall. This practice is frequently resorted to, and the inspector must be alert to prevent it. When the wall is over four feet thick, it is not specified that the headers shall extend through. In this event the header

need go only to the stretcher on the opposite face.

It is sometimes specified that "headers shall hold the size in the heart of the wall that they show in the face." The sketch **a** in Fig. 13 shows the violation of this rule. The stone **A** is represented in elevation, and it is readily seen that pressure from above has a tendency to force it into the dotted position. Sketch **b** shows, in plan, a wedge-shaped stone **B**; and it is apparent that vertical pressure will not force it out of position if it be of uniform depth from front to end. For this reason it is often allowable to use stones of this shape in walls of not very high-class masonry.

Stonecutters often cut away too much from the top and bottom of stones in order that the plane surface marked **c**, Fig. 14, may be more easily prepared. The practice is bad in first-class work, and can be detected only during construction or when the stone cracks on account of the concentration of pressure on too small an area. Such a joint is said to be **flushed.**

A practice even worse is shown at **b** in the same figure. Here not enough has been cut away and the pressure is again on a small area in the interior, and the joint is **open.**

MEASUREMENT OF MASONRY

There are no universally adopted rules for measuring stone masonry, and the custom in

each locality must be ascertained before bidding upon or measuring up work of this kind.

Formerly the unit of measure in masonry was the **perch**, or the volume of a wall a rod ($5\frac{1}{2}$ yards or 16 feet 6 inches) long, a foot and a-half thick, and a foot high, which amounts to $24\frac{3}{4}$ cubic feet. As long as the wall is 18 inches thick,

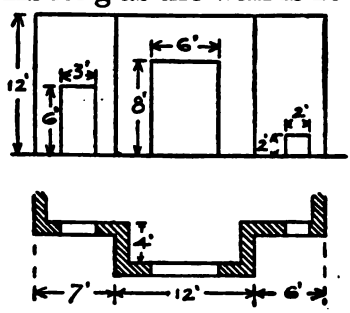


Fig. 15. Measuring Masonry in a Small Building.

the measurement is easy enough, since the product of the length in rods by the height in feet gives the volume in perches at once. For other thicknesses, the linear dimensions were taken in feet, and then divided by the proper number to reduce to perches. It is something of a task to divide by 24.75, and gradually it became the custom to use 25 instead. Of late, however, it is usual to keep the cubic foot as the unit, instead of the perch; and this practice is to be recommended.

It is easier to build a straight, plain wall than one containing corners or curves; and for this reason the number of cubic feet in a wall is not an exact index of the worth of such work. One way of allowing for such extra work is to count

the openings for windows and doors as solid, thus adding to the actual number of cubic feet in the wall. Another rule sometimes adopted is to take out the openings, and add a cubic foot for each linear foot of corners to be finished, including the vertical sides of the windows and doors.

For example, let Fig. 15 represent the plan and elevation of the front of a small building to be measured. According to the first rule, the volume is:

$$(8+25)\times 12\times 1\tfrac{1}{2}=594 \text{ cubic feet.}$$

By the second rule, it is:

$$594-(4+18+48)\times 1\tfrac{1}{2}+12\times 6+32=593 \text{ cu. ft.}$$

If the extras for the corners be counted 18 inches thick, the result will be 52 cubic feet more, or 645 cubic feet.

If the corners are in foundations under ground, they are usually left so rough that no allowance is made for building them.

Usually openings not wider than twice the thickness of the wall are measured as solid; and some allowance is agreed upon as extra to be added for building corners, curves, or other unusual work.

It is seen from the above example, that as much as ten per cent of the total volume may depend upon the method adopted in the measurement. Accordingly, care should be taken that all parties concerned are, from the first, acquainted with the practice to be adopted.

STONE STRUCTURES

The Foundation Bed. The treatment of the site on which masonry is to rest, depends upon the character and weight of the structure to be erected. The experience with similar buildings on like grounds, is the best guide in judging of the bearing power of any soil. Where definite experience of this kind is lacking, care must first be taken to ascertain, by pits or borings, the character of the material underlying the surface of the ground. Then, having determined the weight of the structure to be supported, the engineer exercises his judgment in designing the footings so that the unit-pressures will not be excessive.

This subject was discussed at length in the Transactions of the American Society of Civil Engineers (1905), and the following loads in tons per square foot were recommended as safe:

For soft clay ..... 1 ton per sq. ft.
For ordinary clay, and dry sand and clay 2 tons per sq. ft.
For dry clay and dry sand.....3 tons per sq. ft.
For hard clay and coarse sand.....4 tons per sq. ft.
For firm sand and gravel.....6 tons per sq. ft.

A chimney in Lawrence, Mass., has been standing many years with no apparent settlement. It stands on soft material confined by sheet piling 5 feet long. The sand was covered with concrete 35 feet square and 1 foot thick; and on the concrete is granite seven feet thick.

The weight on the soil is 2,150 tons—or 1.8 tons per square foot, average pressure. The effect of the wind is thought to be such as to make the maximum pressure 2.4 tons. There are instances in Boston, Mass., where soft clay carries from 2½ tons to 4 tons safely. The building laws of the given locality must always be consulted before designing foundations for any building.

Improving the Bearing Power of Soil. Whenever the required load exceeds those given above, means must be taken to improve the character-ter of the soil, or to provide other support. The bearing power of even soft material is much improved by enclosing it with a row of short piles to prevent lateral movement and to make possible the drainage of the enclosed area. If the site be above an outlet for the water, drainage may be provided by means of tile around the outside. This is also an effective way of improving the foundation bed. Again, covering a wet, unstable, foundation bed with concrete or even with sand, to a depth of one or two feet, has a marked effect upon its bearing power. "Smeaton mentions a stone bridge built upon a natural bed of gravel only about 2 feet thick, overlying deep mud so soft that an iron bar 40 feet long sank to the head by its own weight."*

Foundations on Piles. In many cases it is cheaper to drive piles to secure a suitable bearing than to remove the soft material overlying a

*Trautwine's Engineer's Pocket-Book.

proper foundation bed. The safe load that a pile will support has been the subject of some experiments and much speculation; but no unfailing rule has been deduced. The expression designed to show this load that is in most common use is known as the **Engineering News formula:**

$$P=\frac{2Wh}{s+1},$$

in which **P** is the safe load on the pile, and **W** is the weight of the hammer of the pile-driver, both being taken in the same unit, usually tons; **h** is the height in feet through which the hammer falls; and **s** is the penetration in inches of the pile under the last blow. That is, the safe load in tons on a pile is equal to twice the weight of the hammer in tons multiplied by the fall in feet and divided by 1 plus the penetration of the pile in inches under the last blow.

For example, if a hammer weighing 2,000 pounds, falling 20 feet, causes the pile to sink 1.5 inches under the last blow, then the safe load, by the formula, will be:

$$P=\frac{2\times1\times20}{1+1.5}=16 \text{ tons.}$$

A pile under similar actual conditions is said to have been loaded with 7.5 tons, with the result of causing much settlement. Another pile, driven with the same weight and fall, sank 26 inches under the last blow, but carried 7.5 tons safely. The piles under the coal pockets on Lincoln wharf, Boston, Mass., are said to carry 12

tons each, safely; and loads of 40 tons are being carried on single piles elsewhere.

To get the full benefit of piling, the piles should be bound together by a layer of concrete extending from the tops one or two feet down to the earth into which the piles are driven. Fig. 16 shows a section of such construction.

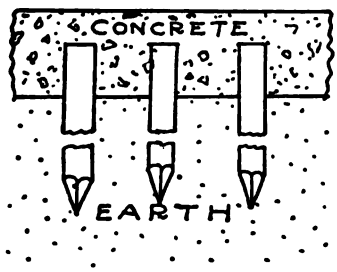


Fig. 16. Use of Concrete in Wooden Pile Foundation.

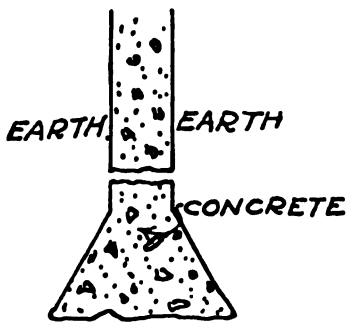


Fig. 17. Simple Method of Concrete Piling.

In Chicago a modification of piling is adopted which consists of digging a series of wells from 3.5 feet to 8 feet in diameter, down to suitable material. At the bottom, these wells are enlarged considerably to secure better support, and the whole well is then filled with concrete. Such a well is shown in Fig. 17. The bed-rock is from 90 feet to 110 feet below the surface, and, of late, the foundations of the heaviest buildings have been carried to this depth.

Mechanics of Beams. Before taking up the design of structures, it will be well to review briefly the mechanics of beams as given in elementary textbooks on the subject.

These formulas are based upon the assumption that the stresses produced are proportional to the loads producing them. This is true only when the beam is not strained enough to be permanently deformed by the load.

The **neutral axis** of a beam is the plane through the center of gravity that is in neither tension nor compression when the beam is bent.

If **W** be the load at the middle of a simple beam supported at the ends; **l**, the length; **b** and **d**, the width and depth in inches respectively; **S**, the stress per square inch; **M**, the bending moment; **I**, the moment of inertia; and **c**, the distance from the neutral axis to the extreme fiber,

$$M = S\,I/c$$
$$M = 1/6\,S\,b\,d^2$$
$$\tfrac{1}{4}\,Wl = 1/6\,S\,b\,d^2$$
$$W = 2/3\,S\,b\,d^2$$

The above equations may be readily applied as follows. If a small beam be assumed as one foot long, an inch wide, and an inch deep, the weight at the middle necessary to break it is given by dividing the values in Table II, for transverse strength, by 18. For example, if this beam be of granite from Milbridge, Maine, the breaking load would be 2,069÷18=115 pounds. This result is not strictly true, since the above relations apply only when the elastic limit of the material is not exceeded. However, by dividing again by a proper factor of safety, the safe load may be found. Thus, dividing the 115

by 4 or 5 shows that the above beam should safely support from 23 to 29 pounds.

In practice, of course, the beam would have other dimensions, and the formula shows that the result obtained above should be multiplied by the breadth and by the square of the depth, and divided by the length. Table V and the accompanying illustrations (Figs. 18 to 23), show the effect of different loadings and ways of supporting the beam.

TABLE V

Transverse Strength of Beams of Uniform Cross-Section

Beam and Loading	Factor	Rectangular Beams		Beams of Any Section	
		Breaking Load	Deflection	Moment of Load	Deflection
As in Fig. 18..	$\frac{1}{72}$	$\frac{1\ Sbd^2}{6\ l}$	$\frac{4\ Wl^3}{E\ bd^3}$	Wl	$\frac{Wl^3}{3\ EI}$
As in Fig. 19..	$\frac{1}{36}$	$\frac{1\ Sbd^2}{3\ l}$	$\frac{3\ Wl^3}{2\ Ebd^3}$	½ Wl	$\frac{Wl^3}{8\ EI}$
As in Fig. 20..	$\frac{1}{18}$	$\frac{2\ Sbd^2}{3\ l}$	$\frac{Wl^3}{4\ Ebd^3}$	¼ Wl	$\frac{Wl^3}{48\ EI}$
As in Fig. 21..	$\frac{1}{9}$	$\frac{4\ Sbd^2}{3\ l}$	$\frac{5\ Wl^3}{32\ Ebd^3}$	⅝ Wl	$\frac{5\ Wl^3}{384\ EI}$
As in Fig. 22..	$\frac{1}{9}$	$\frac{4\ Sbd^2}{3\ l}$	$\frac{Wl^3}{16\ Ebd^3}$	⅛ Wl	$\frac{Wl^3}{192\ EI}$
As in Fig. 23..	$\frac{1}{6}$	$\frac{2\ Sbd}{l}$	$\frac{Wl^3}{32\ Ebd^3}$	$\frac{1}{16}$ Wl	$\frac{Wl^3}{384\ EI}$

For example, a lintel of Troy, N. H., granite 5 feet long, 6 inches deep, and 8 inches wide, having its ends fixed in the walls, carries a uniform load. From Table II, the strength is found to be 2,335 pounds per square inch. In Table V, the

factor is $^1/_6$, which is always to be multiplied by the width and by the square of the depth, in inches, and divided by the length in feet. So the ultimate uniform load is

$$W=\tfrac{1}{2}\times{}^1/_6\times8\times6\times6\times2{,}335=22{,}416 \text{ pounds.}$$

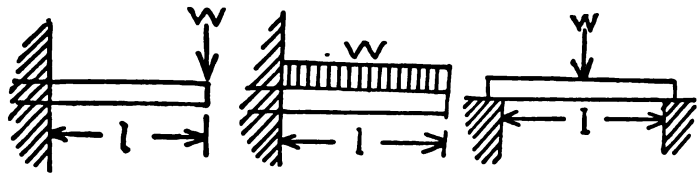


Fig. 18. Fig. 19. Fig. 20.

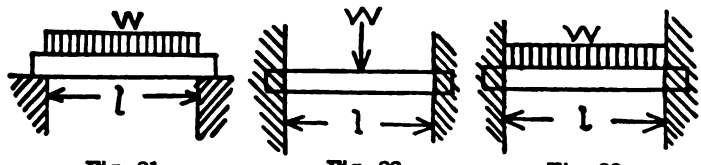


Fig. 21. Fig. 22. Fig. 23.

Different Methods of Loading and Supporting Beams.

For safety this is to be divided by 4, so that the safe load is

$$W=22{,}416\div4=5{,}604 \text{ pounds.}$$

If it be required to find the **deflection** of the same beam, or the **bending moment** of a beam of other than rectangular cross-section, it is necessary to know the meaning of the symbols **E** and **I.** The first letter indicates **coefficient of elasticity;** and the latter, **moment of inertia.**

It is a well-known fact, verified by experiments, that all materials will change in length when loaded; and that this change in length is exactly proportional to the loads, if the latter

be not too large. This fact is known as **Hooke's Law.** If a pound weight be placed on a small piece of rubber, the latter will be seen to change its length materially. If the weight be placed on a piece of iron of the same size, the iron will change its length just as surely, but by a much smaller amount. Now, if **P** be the weight on an area **A**; **l**, the length of the material in the direction of the load; λ, the deformation caused by the load; **S**, the load per unit of area of cross-section; **s**, the deformation per unit of length; and **E**, the ratio of unit-stress to unit-elongation, or the **coefficient of elasticity**, then, from the definitions, $E = S \div s$; $P = AS$; and $\lambda = sl$. Then, by substituting, we have:

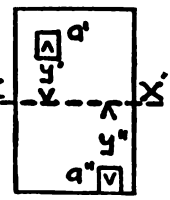


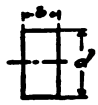
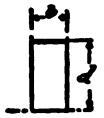
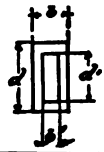
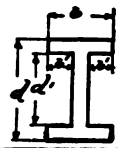
Fig. 24. Illustrating Moment of Inertia.

$$\lambda = \frac{Pl}{AE}.$$

Let the area in Fig. 24 be made up of many small areas as a′, a″, etc., each distant y′, y″, etc., from an axis **xx′**. If each area **a** be multiplied by the square of its corresponding **y**, and the results added together, the sum will be what is known as the **moment of inertia** of the area about the axis **xx′**. The moments of inertia for sections in common use are given in Table VI, the axis being through the center of gravity unless otherwise stated.

TABLE VI

Moments of Inertia of Various Sections

Section I	Section I	Section I
 $\frac{bd^3}{12}$	$\frac{bd^3}{36}$	$\frac{Ad^2}{16}$
 $\frac{bd^3}{3}$	$\frac{bd^3}{12}$	$\frac{Ad^2-ad'^2}{16}$
 $\frac{bd^3-b'd'^3}{12}$	$\frac{bd^3}{4}$	 $\frac{bd^3-2b'd'^3}{12}$

When it is required to find the moment of inertia of a section about an axis, distant **h** from that through the center of gravity, the formula is:

$$I'=Ah^2+I$$

in which **I'** is the desired moment of inertia, **h** is the distance from the required axis to that through the center of gravity, and **I** is the moment of inertia with respect to the latter axis.

For example, let it be required to find the moment of inertia of a rectangle with respect to the base; then,

$$I'=bd\ (d/2)^2+bd^3/12=bd^3/3,$$

as given in the table. The operation is the same if the required axis is entirely outside the figure.

Design of the Footing. The footing courses of a building are so proportioned that the above-named allowable unit-pressures on the soil are not exceeded. In other words, the total weight, divided by the safe load, will determine the total area of the footing. In most cases where the foundation is not on rock, there will be more or

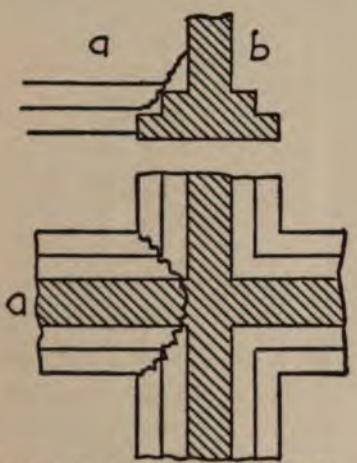


Fig. 25. Design of Footing to Prevent Uneven Settling of Walls.

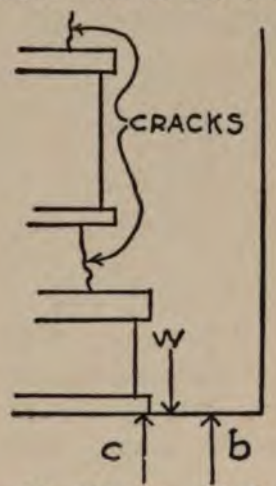


Fig. 26. Pressures and Reactions in Walls and Soils.

less settling; and the best that can be done is to make this uniform over both outside and inside walls. The weight on each wall should be estimated as nearly as possible, and the width of the footings made to correspond.

In Fig. 25 the cross-wall **a** has settled less than the outside wall **b**, and an irregular crack has been occasioned as shown. The remedy is to increase the width of the footing of **b**, or to decrease that of **a**.

The effect of the relative positions of the downward weight and of the upward reaction of the soil, is shown in Fig. 26. It is nearly impossible to make these exactly opposed to each other; but it is manifestly better that the wall **a** have a slight tendency to lean toward the opening than from it. This is the result when the upward reaction is at **b** rather than at **c**, when

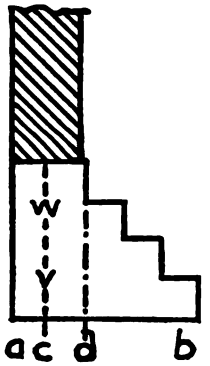


Fig. 27. Eccentric Loading.

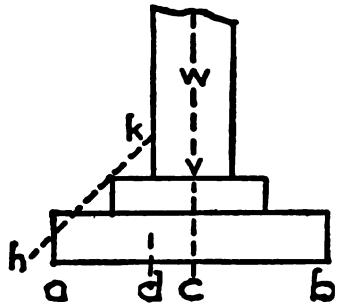


Fig. 28. A Footing Acting as a Cantilever Beam.

the weight of the wall is centered at **W**; and it is clear that the footing should be narrower or altogether omitted under the doors and windows. It is impracticable to state this rule very definitely, since there is always considerable rigidity in the masonry, which makes it impossible to fix the exact point of application of the upward reaction of the soil. However, in general, the ends of a wall should have rather better support than the middle parts; and the outside walls better than those in the interior.

Courtesy of Brown Hoist Co.

TWO BLAST FURNACES, WITH THEIR HOT-BLAST STOVES, AT PLANT OF CRUCIBLE STEEL COMPANY, CLAIRTON, PENNSYLVANIA.

Courtesy of Krupp Works, Germany.

INTERIOR OF A CRUCIBLE STEEL FOUNDRY.

Casting a large ingot of crucible steel.

Eccentric loading. In city lots the space is so valuable that the footings do not extend much outside the walls. The result is that the loads due to the weight of the building are applied as shown in Fig. 27, and the pressure at **a** is greater than at **b**. When **bc** is twice **ac**, the pressure at **a** is twice the average pressure on **ab**, and that at **b** is zero. This question will be more thoroughly investigated below, under the heading "Retaining Walls." The addition of width, as **db**, actually increases the unit-pressure at **a** unless **db** is twice **ad**.

Strength of Footings. In designing footings, it has been the custom to consider the part projecting beyond the pier as a cantilever beam (see Fig. 19). Thus, in Fig. 28, the length of the beam has been taken as **ad**. Experiments prove that the footings often do fail in this place, but through shear rather than bending. If the footing be considered as a cantilever beam (Fig. 28), with a load $\frac{1}{2}$ **W** upon it, the length should be taken as **ac**, rather than **ad** as many do, since the breaking frequently begins at **c** before it does at **d**. It is a good rule to make the slope **hk** 45 degrees.

Strength of Masonry Walls. The strength of the stone of which a wall is built, is not a sure index of the strength of the wall itself. The thickness of the joints, the strength of the mortar, and the thoroughness of the workmanship are as important as the character of the stone itself. The Austrian Society of Civil Engineers

has published the results of tests made on stonework, which go to show that the loads used in practice are well within the limits of safety, if the workmanship be good. The strength of the stone itself was found by crushing blocks 11.81 inches square and 15.75 inches high, with results as shown in Table VII, in pounds per square inch:

TABLE VII

Crushing Strength of Stone

Pounds per Square Inch

Kind of Stone	Signs of Breaking	Failure
Granite	10,860	10,910
Granite	8,180	8,550
Hard Sandstone	5,460	10,380
Hard Sandstone	5,500	8,780
Granite Pier	.....	8,110
Granite Pier	.....	8,310
Hard Sandstone Pier	2,760	3,220
Hard Sandstone Pier	2,810	3,870
Hard Sandstone Pier	2,290	2,780
Hard Sandstone Pier	2,360	2,820

The piers (granite and sandstone) are shown in Figs. 29 and 30; and the manner of loading the granite is shown in Fig. 29. It will be noted that the eccentric load caused the pier to shear as it crushed, and that the strength of the pier was nearly that of the granite itself. The mortar used in the first two piers was 1 part Portland cement to 2 parts sand; and in the third and fourth piers, 1 part Portland cement to 3½ parts sand. The age of the piers was three and a-half months.

Loads on Masonry Walls. The weight of the building itself, including the walls, partitions,

floors, and roof, is called the **dead load.** The weight of people, furniture, goods, machinery, movable safes, the effect of the wind, and anything readily movable, is the **live load** or **movable load.** The total dead load is quite well known by the architects; but the manner of its distribution over the walls and partitions is indefinite, owing

Fig. 29.

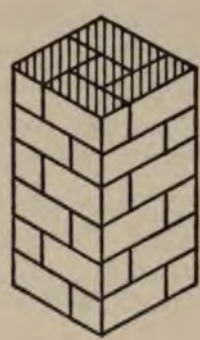


Fig. 30.

Masonry Piers Tested by Austrian Society of Civil Engineers.

to the rigidity of the frame of the building. The amount of the live load, and the route by which it is carried to the partitions, walls, and foundations, can be ascertained only approximately.

In designing footings and walls, it is usual to provide for the live or moving loads, excepting those due to wind, by arbitrarily increasing the known dead loads. The weight of live loads is usually exaggerated unless they are actually weighed. An examination of these loads in the Marshall Field wholesale store in Chicago,

proved them to be but 57 pounds per square foot, while double that amount would usually be provided for.

The Chicago building laws permit a load of 25,000 pounds per square foot on well-surfaced, squared stone masonry set in standard cement mortar. For less carefully surfaced stone, the load allowed is 20,000 pounds per square foot. The same laws provide that for buildings used for the sale, storage, or manufacture of merchandise, the floors shall be so designed and constructed as to be capable of bearing a live load of 100 pounds per square foot of floor surface, in addition to the dead load. The dead load includes the weight of the floor itself, partitions, permanent fixtures, and mechanisms.

RETAINING WALLS

A wall such as that shown in Fig. 31 is called a **retaining wall** because its office is to keep a bank of earth in position. The earth to be retained may be that already in place before the wall was built, as where a cut is made in the side of a hill; or the wall may be constructed, and the earth back of it deposited later.

Granular material, such as sand, coal, grain, and such substances, when unsupported on the sides, will assume a conical shape, and the angle made by the slanting side of each cone and the horizontal is peculiar to each substance, and is called the **angle of repose** of that material. The natural slope or angle of repose of earth is

usually assumed to be such that the base is one and a-half times the height.

In Fig. 31, the earth below the line **cd** will exert no pressure on the back of the wall, and hence the weight of the wedge-shaped mass above that line only is to be considered. If the material back of the wall be liquid, the line **cd** is evidently horizontal. Water is the only ma-

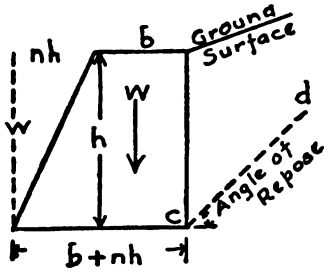


Fig. 31. Detail of a Retaining Wall.

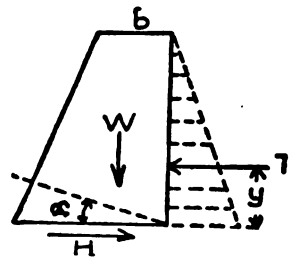


Fig. 32. Forces Tending to Cause Sliding of a Retaining Wall.

terial with which exact computations of the pressure against a wall can be made, since the angle of repose of other material changes with the degree of moisture in it. Such an investigation of the pressure of water would belong rather to a discussion of the subject of **Dams**, and will not be presented here. Although the computation of the pressure of earth is unsatisfactory, experience has taught some rules that lead to safe design of retaining walls.

There are three ways by which a masonry retaining wall or dam may fail, even if it be not undermined or disintegrated—namely:

(1) By sliding bodily along its base or along any horizontal joint;

(2) By overturning about its outer edge, either at the base or elsewhere.

(3) By the crushing of the outer edge, reducing the area of the base, and causing overturning.

Stability against Sliding. In Fig. 32, let the pressure against the back of the wall be **T**; the weight of the wall be **W**; the horizontal resistance to slide be **H**; and the coefficient of friction

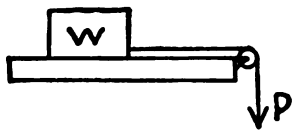


Fig. 33. Illustrating Coefficient of Friction.

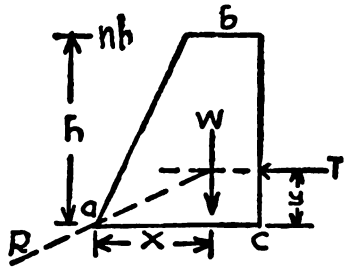


Fig. 34. Designing a Retaining Wall against Rotation.

be **f**. Then, for equilibrium, **H**=**T**=f**W**; also, **H** cos a=**W** sin a; and **H**=**W** tan a. Hence, if **T** be constant, the stability is improved by slanting the courses upward from the back toward the front, or by increasing the weight **W**.

If a force **P** be sufficient to cause a weight **W** to move along a horizontal plane, then P÷W is said to be the **coefficient of friction** between the two substances (see Fig. 33). Again, if the weight be on a plane that is gradually inclined to the horizontal, it will become more and more

insecure, and will finally begin to slide. If the angle at which sliding begins be **a**, then tan **a** is also equal to the coefficient of friction. This is usually stated: "The coefficient of friction is equal to the tangent of the angle of repose." The coefficients of friction for some common conditions are given in Table VIII.

TABLE VIII

Coefficients of Friction

Rough granite on wet sand	0.41
Rough granite on dry sand	0.65
Masonry on dry or clayey gravel	0.51
Masonry on moist clay	0.32
Point-dressed granite on moist clay	0.70
Fine-pointed granite on moist clay	0.58
Common brick on moist clay	0.64

If there is any danger of sliding, a wall may be made more secure by using mortar in the joints, and sinking the wall into the earth.

From the above equations, it is seen that, so far as sliding is concerned, the shape of the cross-section of the wall is immaterial.

Stability against Rotation. In Fig. 34, the wall is in equilibrium as to rotation when $\mathbf{W}\,\mathbf{x} = \mathbf{T}\,\mathbf{y}$. If the height of the wall be fixed, **T** and **y** must be known or assumed; then the problem is to proportion the weight and its point of application so that **W x** will be large enough. The weight is increased by making the wall thicker or of heavier material. The distance from the toe of the wall to the vertical through the center of gravity, is increased, while the volume of the wall remains constant, by making the top nar-

row, the base wide, and a heavy batter on the front.

If failure by overturning were the only thing to consider, a triangular cross-section, as **a b c** in Fig. 35, would be proper; but a material thickness at the top must always be provided to withstand shocks, the action of frost, etc. For this

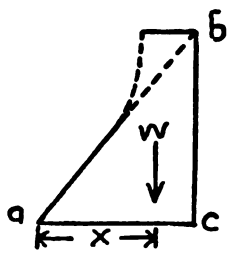


Fig. 35. Method of Strengthening Top of Retaining Wall.

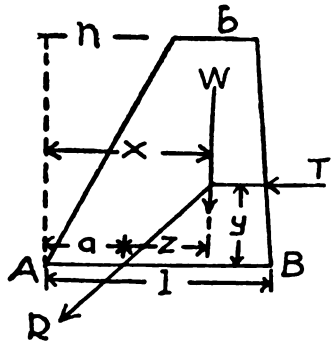


Fig. 36. Designing a Retaining Wall against Crushing.

reason, a wall sustaining a bank of earth is usually built with a top thickness of at least two feet, and little or no batter on the back. This design makes **x** large and promotes stability. Where the available space for the base of the wall is limited, as along the edge of a city street, the batter must be on the side toward the bank. In this case, to make **x** as large as before, the wall must be made thicker.

Stability against Crushing. Where two forces like **W** and **T** in Fig. 36 act at one point, the effect may be combined in one **resultant,** which takes the place and has the effect of the

two. Thus, in Fig. 36, **W** and **T** are replaced by their resultant **R**, which pierces the base at some point to the left of **W**. This distance (**a**) depends upon the relative magnitudes of **W** and **T** and the height of **T** above **A B**. If **a** be half of the base, the pressure is evenly distributed over the length **A B**; but as **a** becomes smaller, the pressure at **A** becomes larger, and that at **B** becomes correspondingly less. It can be proved that when the resultant is one-third of **A B** from **A**, the pressure at **A** is twice the average pressure, and that at **B** becomes zero. At this time the joints on the back of the wall are just upon the point of opening, and, unless the wall is held together otherwise than by gravity, it is in an unstable condition.

In the design of gravity walls, it is an invariable rule to keep the resultant in the middle third of the base. When **a** is greater than one-third of the base, the unit-stress at **A** is:

$$S=\frac{2\,W\,(2\,l-3\,a),}{l^2}$$

in which **S** is the stress per unit of area, and **l** is the length **A B** or the base of wall.

When **a** is less than one-third of **l**, the stress becomes:

$$S=\frac{2\,W}{3\,a}.$$

When **a** is just one-third of the base, the results will be the same by either formula.

These formulas show that, for a given weight

of wall and a given thrust, **a** must be made large by having the top as narrow as possible and most of the batter on the outside.

The value of **a** is found as follows: The distance **x** from **A** to the vertical through the center of gravity of the cross-section is given by the formula:

$$x=\frac{n\,b+b^2}{3(b+l)}+\frac{n+l.}{3}.$$

Then, in the triangle formed by **W**, **R**, and **A B**,

$$z:y=T:W$$

Then **x**-**z**=**a**, and the point where **R** pierces the base is known.

To compare the several modes of failure, let a wall be assumed of stone masonry weighing 150 pounds per cubic foot; let the back be vertical, and the thrust against it 10,000 pounds for each linear foot of wall, applied 10 feet above **AB**; and let the height, top, and base be 30 feet, 4 feet, and 12 feet respectively. Then,

W=36,000 pounds per linear foot

$$x=\frac{8\times4+4\times4}{3(4+12)}+\frac{8+12}{3}$$

$x=7\frac{1}{3}$ feet

$z=10{,}000\times10\div36{,}000=2'/_{9}$

Then $a=4^{8}/_{9}$ feet.

Then for **sliding**, taking the coefficient of friction as 0.6, the factor of safety is $36{,}000\times0.6\div10{,}000=2.16$.

For **overturning,** the factor of safety is $\mathbf{W}\,x \div \mathbf{T}\,y$, or $36{,}000 \times 7\frac{2}{3} \div 10{,}000 \times 10 = 2.76$.

For **crushing,** $S = 72{,}000(24 - 14\frac{2}{3}) \div 144 =$ 4,667 pounds per square foot, while the stone would sustain 100 times as much. It might be assumed that the factor of safety is 100; but such is not the case, for, if the thrust be increased to 13,000 pounds, the joints at the back are upon the point of opening, letting in water and creating entirely different conditions, and the real factor of safety is only about 1.3, instead of being very large. So it is seen that an investigation of the unit-pressure on the base is important.

Safe Thickness for Retaining Walls. The actual pressure of earth against retaining walls is not well known; but experience shows that it is not safe to design walls with a thickness on top of less than 2 feet, and with a base of less than 0.4 of the height. Some high retaining walls in the city of Quebec have failed after standing for many years. The heaving action of frost is mainly responsible for these failures. Once a wall is forced forward, ever so little, it seldom regains its proper position; and each year finds it farther out of plumb, until its equilibrium is destroyed.

In building walls to retain banks of earth already in place, it is a common saying among workmen of experience: **Never let the bank begin to move.** This applies to the work during construction, and implies that the bank when first cut has a stability, due to the adhesion of

its particles, which it does not long retain. If it is allowed to begin to slide, it is very difficult to hold it with falsework, and the wall must be made heavier than would otherwise be necessary.

In Louisville, Ky., the sliding of a large retaining wall was effectively stopped by erecting a barrier in front of the sliding wall, consisting of another wall of reinforced concrete on piles, and filling in back of same.

A Dam Failure. Fig. 37 shows the cross-section of a masonry dam on the Colorado river above Austin, Texas, which failed April 7, 1900,

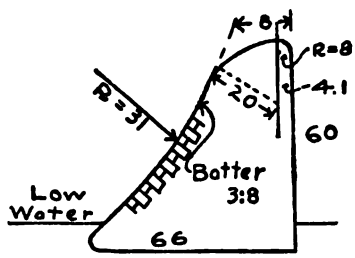


Fig. 37. Cross-Section of Dam at Austin, Texas.

letting loose a reservoir of water 30 miles long, half a mile wide, and 50 feet deep. At that time the flood water was so high as to cause a thrust of perhaps 200,000 pounds on each foot of the dam. From the above investigation, it is probable that this pressure was not enough to cause failure by any of the three ways enumerated. A subsequent investigation showed that the foundation of the dam had become impaired on the down-stream side, the dam settled and cracked,

a part of it was overturned, and part was moved bodily down stream.

THE MASONRY ARCH

The **arch** is a structure used primarily as a device for covering openings too long to be spanned by beams of reasonable dimensions. Masonry of all kinds is much stronger in com-

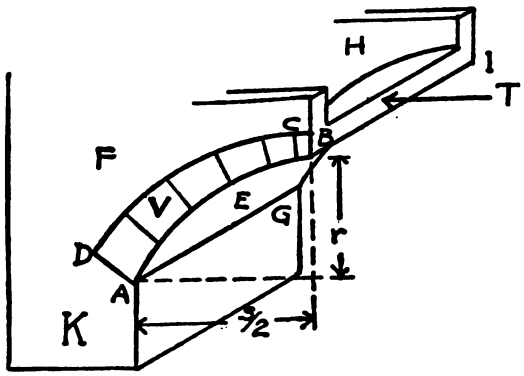


Fig. 38. Half-View of an Arch, Showing Parts.

pression than in tension; and by the use of the arch, stresses of the latter variety need not be used.

Fig. 38 shows half an arch, and the parts represented are named as follows:

Keystone ..... The stone at B C.
Arch-Ring ..... The stones at A D C B—I G.
Intrados ..... The curve, as A B, on a section parallel to A D C B.
Extrados ..... The curve, as D C, on a section parallel to A D C B.

Crown ..... The part of the arch-ring near C.
Haunches ..... The part of the arch-ring near V.
Skewback ..... The surface A D G on which the arch rests.
Abutment ..... The wall K below the skewback.
Soffit ..... The surface A B I G.
Spandrel ..... The space between the arch and the roadway.
Spandrel Filling.. The material H within the spandrel, to support the roadway.
Spandrel Wall... The masonry wall F.
Springing Line... The line AG.
Voussoir ..... The separate stones of the arch-ring.
Span ..... The width of opening S.
Rise ..... The height (r) of B above A.

Arches are designated according to the shape of the intrados, as **circular, elliptical,** etc.; according to the angle included between the radii at the springing line, as **full-centered,** if the angle be 180 degrees, and **segmental,** if the angle be less; as **right arch** or **skew arch,** according as the axis of the cylinder forming the soffit is at right angles or otherwise to the face or spandrel wall **F.**

Investigation of the Arch. As is the case with walls and dams, arches fail oftener by set-

tlement and by being undermined than by methods of failure that can be investigated analytically. Assuming that the abutments and foundations are secure, arches fail, as do retaining walls, either (1) by the sliding of one stone upon

Fig. 39. Arch Too Heavily Loaded at Crown.

Fig. 40. Arch Too Heavily Loaded at Haunches.

another; or (2) by the rotation of one of the stones of the arch-ring about its edge; or (3) by the crushing of one of the stones of the arch-ring.

(1) **Failure by Sliding.** If the arch be too heavily loaded at the crown, as in Fig. 39, the part **B** has a tendency to settle; and other parts, as **A** and **C**, may be forced outward if the abutments are immovable. If, on the other hand, as in Fig. 40, there be relatively too much load at **A** or **C**, these parts settle, and **B** may be forced upward. These results are accomplished only when the component of the resultant parallel to a joint is greater than the normal pressure times the coefficient of friction, or when **H** is greater than **Nf** in Fig. 41. The values of **f** for various materials are found in Table VIII.

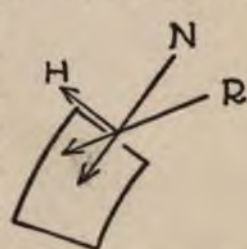


Fig. 41. Reactions in an Overloaded Arch.

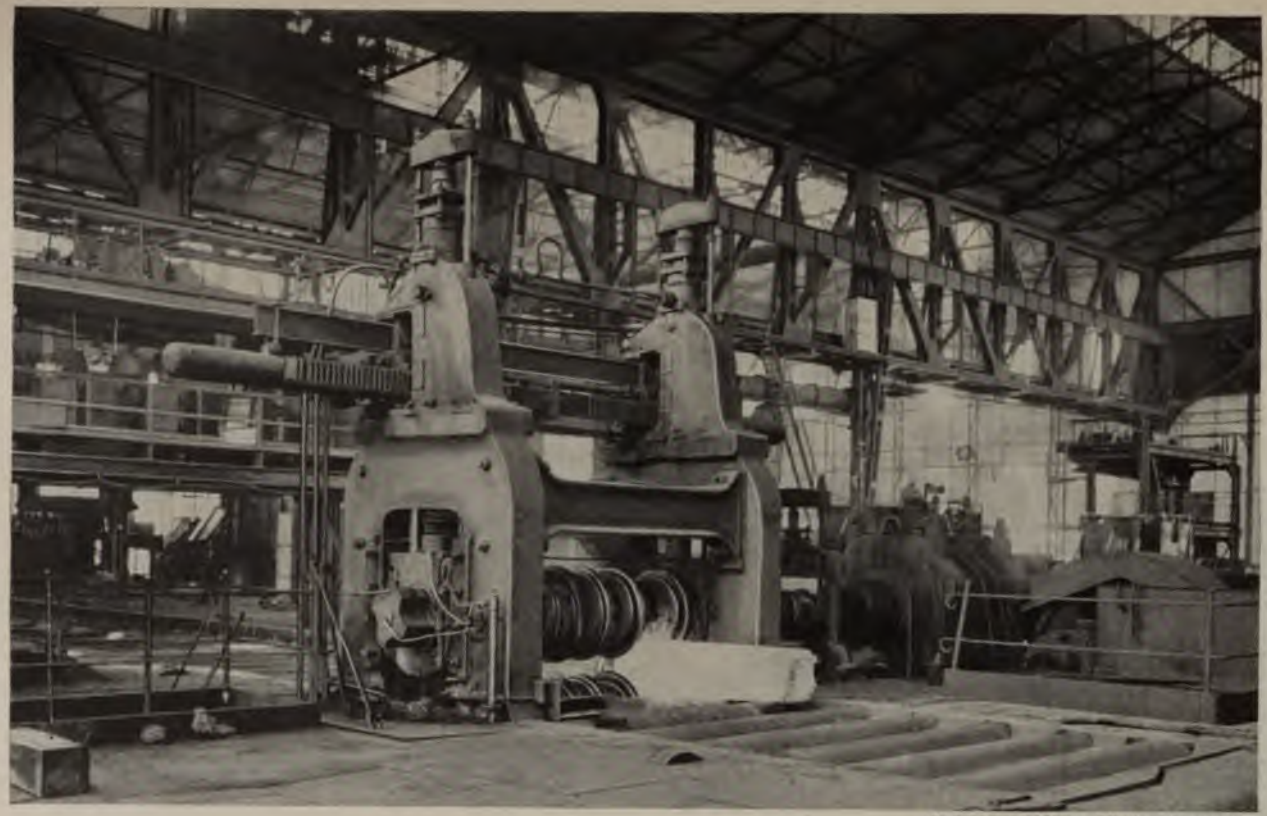
The factor of safety against sliding is **Nf÷H.** Apparent cases of such sliding as is shown in Figs. 39 and 40—common in arches over doorways in buildings—are often caused by the spreading of the sides of the doorway, rather than by excessive loading originally.

(2) **Failure by Rotation.** By reference to the investigation of retaining walls, already given, it will be seen that rotation occurs only when the resultant pierces the base at or beyond the limits of the area. The tendency to rotate depends upon the position of the resultant pressure, or upon **a** in Fig. 36. When **a** is one-half of **L,** there is no tendency to rotate; and when **a** is zero, the equilibrium is unstable, and rotation is about to take place. In general the factor of safety against rotation is given by:

$$\text{Factor of safety} = L \div (L - 2a).$$

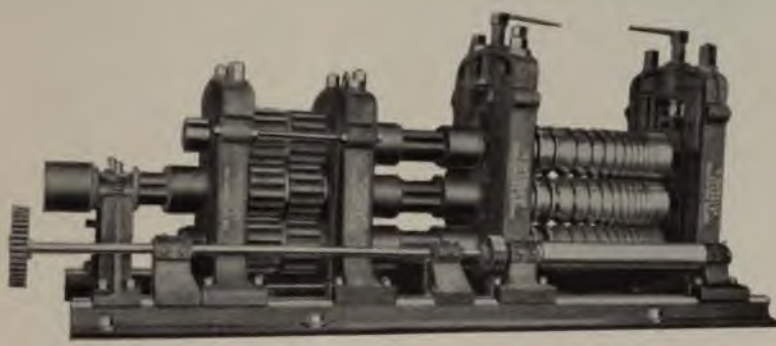
When **a** is one-third of **L,** the factor of safety is 3 against overturning.

(3) **Failure by Crushing.** When the magnitude, direction, and point of application of the resultant of the forces above any point are known, the investigation of the crushing effect is like that in the case of the retaining wall, and need not be repeated. It should be kept in mind that the crushing is due to the eccentricity, as well as to the magnitude, of the load. If the outer edge be broken by excessive pressure or by the action of water and ice in the joints, the area over which the load is distributed is

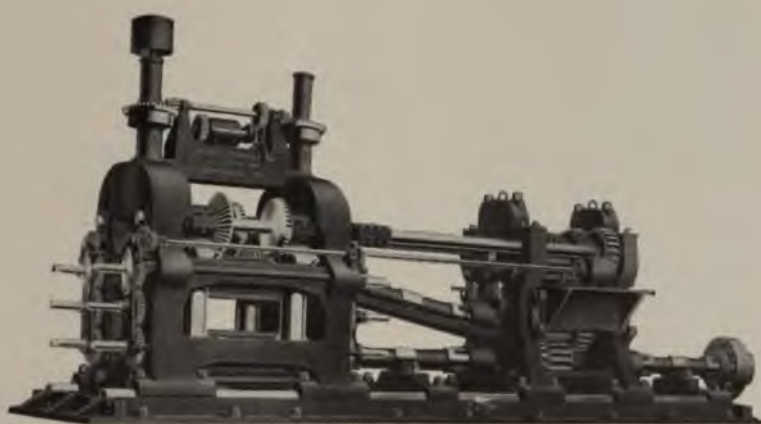


Courtesy of Krupp Works, Germany.

INGOT PASSING THROUGH ROLLS IN A BLOOMING MILL.



A 16-INCH "THREE-HIGH" MERCHANT MILL.



Courtesy of United Engineering & Foundry Co., Pittsburgh, Pa.

A 48-INCH UNIVERSAL MILL.

PLATE 5—STEEL CONSTRUCTION.

reduced, and failure may take place by rotation or by further crushing.

External Loads on Arches. Whether the arch be intended to support the masonry over a doorway; or the spandrel filling, road foundation, and pavement, with the traffic; or a railroad track or train, there are very many uncertainties as to the amount and distribution of the load. The usual assumption regarding the effect of a brick wall above an arch, is that the arch should be designed to carry the weight of a triangle of the masonry, whose length is the span, and whose height is half the span. If the arch be flat, this load may be considerable.

The weights of the filling, pavement, track, and moving loads may all be determined with considerable accuracy. The uncertainty as to the way in which these weights act upon the arch, makes necessary some assumptions which may not be exactly true.

If the load is of sand, or is carried to the arch through considerable thickness of sand, it is reasonable to suppose that it exerts some horizontal pressure. If, on the other hand, the loads are carried down by a spandrel filling of loose rubble masonry, the horizontal pressure must be small.

Experimental Tests on Arches. Perhaps the most notable tests of arches were made by the Austrian Association of Engineers and Architects during 1900-1905. The following account of the results is abstracted from "Engineering" (London, February 21, 1896):

The long arches tested were 6.65 feet wide, and 74.5 feet span, with a rise one-fifth the span. They were built in a quarry, between solid rock walls so that all deflections were due to the effect of the load on the arch. A platform supported on six sets of columns, the feet of which rested directly on the extrados of the arch, extended in each case from one abutment to the crown; and the testing was effected by piling rails on this platform.

The first experiments were made upon an arch of cut stone, and upon one of brick. The stone used was a fairly hard limestone of excellent quality. The voussoirs of both arches were 1.97 feet thick at the crown, and 3.6 feet deep at the springings. The mortar used was mixed in the proportion of 500 pounds of Portland cement to 35 cubic feet of clean sand.

The stone arch gave way when the load piled on the platform reached an amount equivalent to 1.99 tons per foot run; and the brick arch, when the load reached 1.81 tons per foot run. Up to the point of rupture, the stone arch gave no signs of incipient failure; but, in the case of the brick arch, cracks declared themselves previously, which were apparently caused by the failure of the mortar, the bricks themselves being intact.

The arches were 51 days old when they were broken. In failing, radial cracks appeared on the extrados of the loaded side, near the skewback, and on the haunch of the unloaded side.

The conclusions were that such masonry arches will safely bear a load causing a stress of from $^1/_{10}$ to $^1/_4$ the ultimate crushing strength.

DESIGN OF ARCHES

Thickness of the Arch-Ring. It is quite possible to design a masonry arch with considerable accuracy; but many are built without computing the stresses. An arch that has stood the test of

time is taken as an example of safe dimensions, and others of similar span and rise are fashioned after it. Nearly all small arches are built stronger than necessary, as it is cheaper to do so than to cut the stone very accurately to just the dimensions necessary. Few arches are built with less than one foot for the depth of the arch-ring.

Trautwine's rule for the thickness of arches of first-class masonry is: t = Square root of $(r+\frac{1}{2}s)$, divided by 4, and with 0.2 added, in which t is the depth at the crown, r is the radius of curvature of the intrados, and s is the span, all dimensions being in the same unit of measurement.

If the masonry be second-class, the depth given by this formula should be increased by one-eighth; and if of soft brick, by one-third. For example, let the span be 50 feet and the radius be 30 feet; then, by the rule, $t = 0.2 + \frac{1}{4}$ square root of $(30+25) = 2.05$ feet. The thickness is usually increased from the crown to the skewback.

Dimensions of Existing Arches. Table IX gives data regarding some arches that are still standing after years of service. While it is evident that they are strong enough, it is not definitely known how much lighter they might have been built with safety. The above formula gives 1.8, 2.47, 3.48, and 4.18 feet respectively for the proper depth of arch-ring for the four spans of Table IX.

TABLE IX

Data Regarding Various Existing Arches

Location	Span	Rise	Radius at Crown	Thickness	
				Crown	Springing
Chesapeake & Ohio Canal, Rubble..	40 ft.	15 ft.	21 ft.	2.00 ft.	.....
Philadelphia & Reading R. R.....	72 ft.	16.5 ft.	47 ft.	2.75 ft.	2.75 ft.
Waterloo Bridge, London, Granite..	120 ft.	32 ft.	112 ft.	4.50 ft.	8.00 ft.
Cabin John Bridge, Washington, D. C., Granite.....	220 ft.	57 ft.	134 ft.	4.00 ft.	6.00 ft.

Thickness of the Abutment. There are several empirical rules for designing thickness of abutments. The one here given is credited to

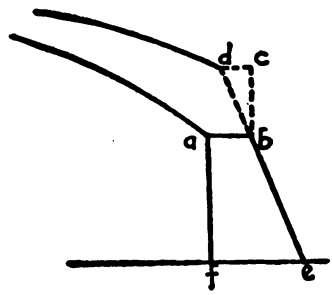


Fig. 42. Finding Batter of Arch Abutment.

the author of Trautwine's Engineer's Pocket-Book.

Let **t** be the thickness of the abutment at the springing; **r**, the radius; and **h**, the rise of the arch—all in feet or in the same unit. Then,

$$t=0.2\ r+0.1\ h+2.0.$$

This formula is applicable to large and small arches alike, and is thought to be safe whether the abutment stands alone or has a backing of earth on the side away from the arch.

Having computed the proper thickness at the springing, the batter of the abutment is found as shown in Fig. 42. The thickness **a b** is laid off; then **b c** is half the rise, and **cd** is $^1/_{24}$ of the span. This batter is produced downward to the bottom of the abutment, and the point **d** is joined with the extrados at the crown by a curve. The width of the bottom **ef** must not be less than $^2/_3$ **af.**

For example, let the height of the abutment of the second arch in Table IX be 12 feet. Then the batter on the back will be $12 \times 3 \div 8.25 = 4.36$ feet; and the width of the bottom will be, by the rule, $4.36 + 2.47 = 6.83$ feet; but the second requirement makes it necessary to build it $^2/_3$ of 12. or 8 feet wide.

Brick Masonry

Bricks have been used as building material for many centuries, and no known substitute seems liable to make them less popular. They are durable, of good appearance, and comparatively cheap. The fact that brick masonry is but little affected by the chemical agents that often destroy stone is well known. As heat is used in the manufacture of brick, such material is well able to withstand the attack of fires that would destroy granite. While stone is no doubt preferred by many for its appearance, very pleasing effects may be obtained from the use of brick, and at considerably less expense.

Brick may be defined as a variety of artificial stone produced by baking clay. While made in a great many shapes and sizes, the blocks are generally of rectangular cross-section approximately 8¼ by 4 by 2¼ inches on the three sides. These small and regular dimensions are factors that make handling, transportation, and laying comparatively easy. No slow and expensive dressing is required; no heavy hoisting apparatus is necessary for loading; and the separate loads can be made greater or smaller as occasion requires.

Composition of Brick. A pure clay, or one containing only **silica** and **alumina**, is seldom found, and, if used, would become distorted and

cracked in the process of burning. The clays used in making brick usually contain some **lime, magnesia,** and **oxide of iron,** together with small proportions of many other substances not especially desirable. Silica, if not in too great proportion, makes the brick burn with less warping than would otherwise be the case. Too much uncombined silica (more than 25 per cent) tends to make the product brittle. Lime has a bad effect by causing the brick to change shape in the burning and to show a tendency to disintegrate afterwards. Iron produces strength, and its presence is indicated by the red color. Usually the red brick are—like the red varieties of stone—the stronger.

To produce **firebrick** a more nearly pure clay is necessary. Iron is in this case not desirable; and not more than 6 per cent of lime, soda, potash, and magnesia combined should be used. Silica may be in excess if the property of resisting heat be the only consideration.

Color of Brick. The color depends upon several factors, which are as follows:

(a) The composition of the clay.
(b) The amount of heat applied in burning.
(c) The sand sprinkled over the outside of the wet brick.

(a) In general, oxide of iron is the controlling factor in determining the color of the brick. Clay usually contains but a very small proportion of iron; and this, upon burning, produces

the dull reds that are characteristic of common brick. If oxide of iron is present in larger percentage—say 10 per cent—the darker blue-black shades result. Lime with iron produces a lighter brown or cream color; and magnesia has a somewhat similar effect.

(b) Clay containing constituents tending to produce a red color will become darker as more intense heat is added. Heat alone will not produce this effect, as is shown in the light-colored firebricks.

(c) In some processes of manufacture, sand is applied to the forms or moulds, to prevent the wet paste from sticking to them. If, as often happens, this sand is red, the brick that might otherwise be of light tint takes the color of the sand rather than that of the clay, and the outside is red.

The clay in this country suitable for brick-making, is mostly glacial material that has been washed down by the streams and redeposited, while its character has undergone little or no change. This is called **alluvial clay**, and the larger pebbles and grains of gravel have been left behind by the stream while the fine clay was in suspension.

MANUFACTURE OF BRICK

Methods. Most of the brick in common use are made of soft paste of clay and water. When water is used very freely, the manufacture is said to be by the **wet** or **soft-mud process.**

When the clay is not so wet, the **stiff-mud process** is used; and when only 4 per cent to 6 per cent of water is in the clay, the method is known as the **dry-press process.** In this last process, the clays are sometimes actually dried rather than dampened before being made into brick.

Many of the small brick-making plants use the wet or soft-mud process, on account of the resulting plasticity and ease with which the clay is moulded. The best results, however, cannot be secured by this method; but some very serviceable brick are thus made.

The larger and better-equipped plants use a stiffer mixture, requiring considerable force to fashion the clay into the desired forms. As compared with the "wet" process, this method requires less water to be taken from the mud brick, and is more economical of time and fuel.

Moulding and Drying. In the dry-press process, the clay is forced into chambers, and compressed by a plunger under heavy pressure, 75 tons or so being applied to the side of a brick.

In the stiff-mud process, the clay is forced from the moulds by a screw or auger mechanism, in a continuous strip. This strip of clay is carried along on a belt or other type of conveyor, and is cut at proper intervals by fine wires passing through it while it is still in motion.

In using the soft-mud process, the wet clay

is forced by a plunger into a set of moulds. As the plunger is raised, the clay on the top of the moulds is struck off, the full moulds are withdrawn, and the empty ones put in their place before the succeeding descent of the plunger. To prevent the sticking of the mud to the moulds, the latter are passed through sand at frequent intervals.

The latter two methods merge into each other, and the above description of the moulding is the usual but not unvarying custom.

Re-pressed bricks are made of soft mud that has been partially dried and then re-formed under great pressure. These bricks are very uniform in shape, but are not particularly strong or hard.

Drying. As the brick comes from the mould, it may contain as much as 30 per cent of moisture. To subject it immediately to intense heat would cause steam to form, and explosions, warping, and cracking would ensue. In order to avoid these results, the bricks are dried for several days on stacks or shelves which allow free circulation of air. The drying may be hastened by surrounding the stacks with steam pipes, which may be arranged according to many different designs. After most of the moisture has evaporated, the bricks are removed to the kilns and arranged for burning.

Burning. The clay is burned to dry it out, and so deprive it of its plasticity; and also to cause partial fusion of the silica and to produce

chemical combination of the various constituents. Burning proceeds in two or three stages—namely, (a) the **slow** or the **water smoking period**; (b) the **rapid stage**; and (c) the **vitrifying stage.**

(a) During the water smoking, the temperature of the applied heat is slowly raised to from 250 degrees to 350 degrees Fahrenheit, to dry out the free water. The finer the clay, the longer the time required for completing this stage.

(b) In the second stage, the heat is rapidly raised to 1,300° F., at which temperature the carbon of the clay unites with the oxygen of the air and passes off. The heat is maintained at this point until all the bricks at both the top and the bottom of the kiln are oxidized.

(c) In making **paving brick,** or that used in sewer work, the heat is usually increased until the outside appears glassy and the brick is hard and nearly impervious to water; this is called **vitrification.**

The burning is by far the most difficult part of brick-making to learn, and the most important part to know.

Kilns used for burning brick comprise a variety of types, which may be classified as follows:

(1) **Up-Draft Kilns;**
(2) **Down-Draft Kilns;**
(3) **Continuous Kilns.**

Semi-continuous and **tunnel kilns** are terms also used, but these latter kilns embrace some of the peculiarities of the others.

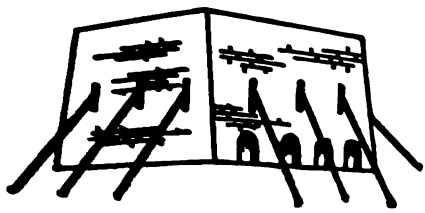


Fig. 43. Old-Style Up-Draft Kiln.

(1) **The Up-Draft Kiln.** The old style of kiln consisted of a pile of mud brick built in the form of arches covering the fire. These piles were surrounded by temporary brick walls,

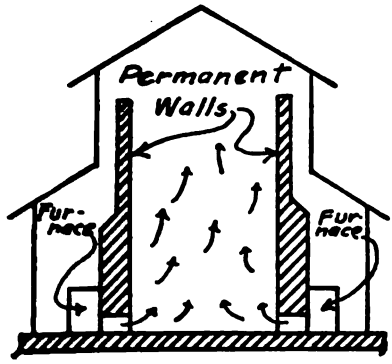


Fig. 44. Scove Kiln Enclosed in Shed.

held in position by braces, which were demolished when the burning was completed. A primitive kiln of this kind is shown in Fig. 43. Later designs have permanent side walls containing the furnaces. These are called **scove**

kilns. The draft is controlled by a course of loose brick at the top of the pile, from which some of the brick may be removed and replaced at will.

Much heat is lost by radiation through the walls, and the burning is not generally uniform throughout the pile. Such a kiln enclosed in a shed is shown in cross-section in Fig. 44.

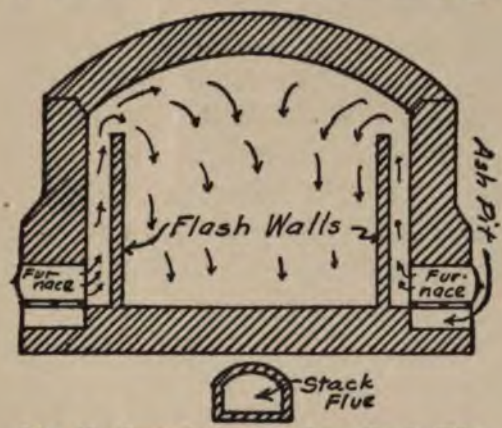


Fig. 45. Cross-Section of Down-Draft Kiln.

(2) **The Down-Draft Kiln.** In this style of kiln, the heat is brought from the furnace to the top of the pile by ducts which enlarge so as to distribute the heat uniformly over the top. This result is more readily accomplished if the roof is dome-shaped as shown. The draft from the chimney carries the heated air downward through the pile and out through the holes in the floor. A cross-section of such a kiln is shown in Fig. 45. The arrows indicate the course of the heated air. These kilns are more

economical to operate, and the brick are evenly burned.

(3) **The Continuous Kiln.** This variety is a series of kilns in which the temperature is found, successively, in all stages from the heat of the furnace to that of the outside atmosphere. The cold air enters through the coldest kiln, and passes through all to the hottest, taking some heat from each.

Sand-Lime Brick. About the year 1880, some German experimenters discovered the fact that brick made of sand and lime could be hardened in a few hours by heat and the pressure of steam. At that time they had no vessels of sufficient size and strength to withstand the required pressure, and the patents were allowed to expire before the process became a commercial success.

Within the last ten years many plants in America have been equipped to make brick in this way, and the product finds a ready sale. Sand-lime bricks are usually light in color, since they are made of nearly white materials; but they are readily tinted any desired shade by adding colors to the water with which they are mixed. Such bricks are very uniform in shape and texture, as there is nothing in the process of manufacture to make them warp or crack. It is claimed that age improves the strength and hardness of sand-lime brick, and that they are strong enough for any ordinary work. The makers are constantly improving their product;

and the latest tests that are at hand show that these bricks are strong enough for facing and other uses where extreme strength is not required. Table X gives data regarding the strength, absorption, etc., of sand-lime brick.

TABLE X

Sand-Lime Brick

Modulus of Rupture Pounds per Sq. In.	Compressive Strength Pounds per Sq. In.	Absorption Per Cent
509	4,844	10.81
766	6,128	8.97
420	2,412	12.70
607	2,244	13.13
555	3,340	9.40
635		8.70

Each of the numbers in the table is the mean of several tests. The sequence of the numbers is of no significance; that is, the numbers in the first line are there by chance.

The modulus of rupture was obtained by breaking the brick, placed on edge between supports six inches apart, the load being in the middle.

The compression pieces were half-bricks, imbedded on both sides in plaster of Paris.

The absorption test was made by heating the bricks to dry them out, and then keeping them in water for forty-eight hours. Upon being taken from the water, they were wiped dry and then weighed; and the increase in weight was divided by the weight of the dry brick, and the result multiplied by 100.

It is seen, by comparison with the results of tests given in Tables XI and XII, that the results for sand-lime brick are about what would apply to rather soft or salmon brick as regards strength, while the rate of absorption is somewhat less for the sand-lime brick. On many accounts such brick are very much better than clay brick of equal strength, as they are of finer appearance, and they may be used for the facing of interior walls such as hallways, corridors, vestibules, and so forth.

CLASSIFICATION OF BRICK

Bricks are classified (1) according to the manner of moulding; (2) according to the place they occupy in the kiln during burning; and (3) according to their shape or the use to which they are to be put.

(1) When distinguished according to the manner of moulding, bricks are named:

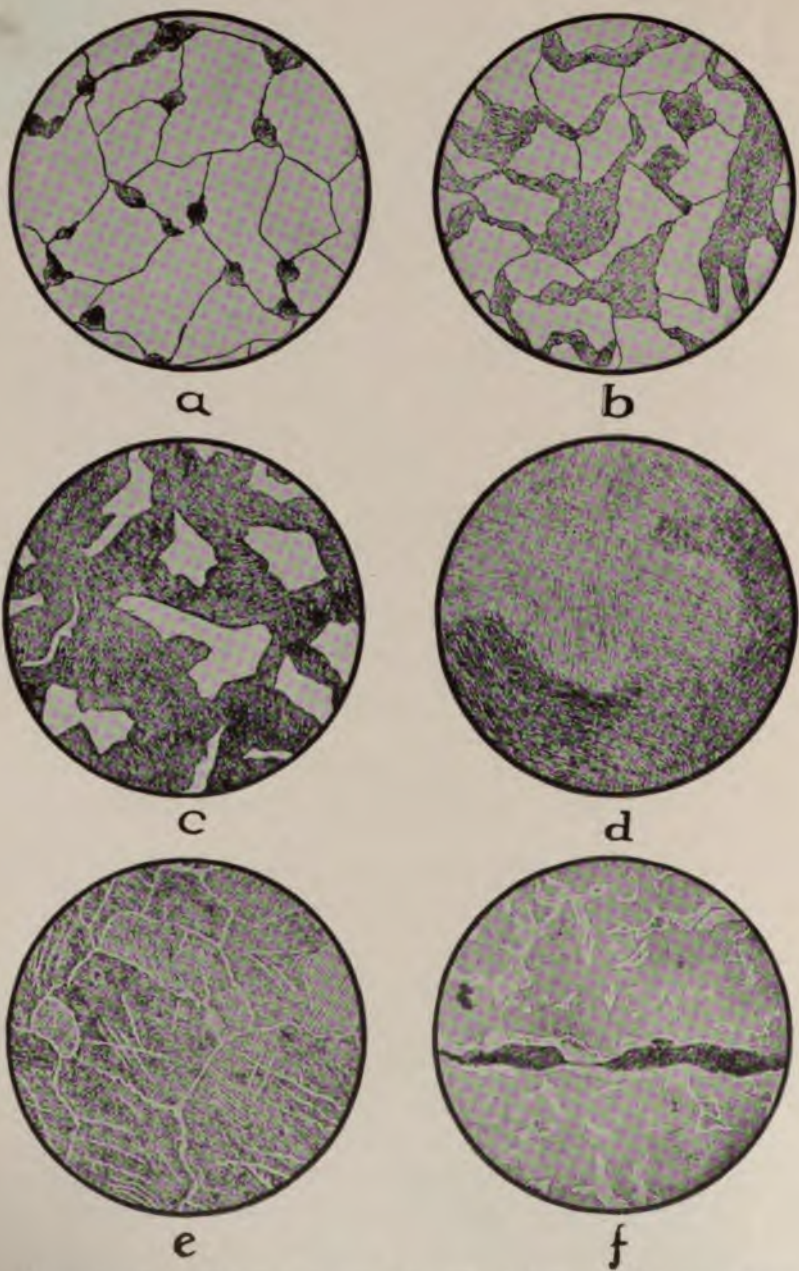
Soft-mud brick, made of very wet mud;

Stiff-mud brick, made of mud with less water;

Dry-pressed brick, made of clay containing four to six per cent of moisture;

Re-pressed brick, made by re-forming partially dried brick made of soft mud.

(2) In the old-style kilns the bricks were not all burned alike. Those adjacent to the fire were over-done; those farthest away from the fire were under-done; and only the intermediate



MICROSCOPIC VIEWS OF STEEL CONTAINING VARIOUS PERCENTAGES OF CARBON OR OTHER METALLOIDS.

Courtesy of American Bridge Co.

CANTILEVER BRIDGE OVER NIAGARA RIVER AT NIAGARA FALLS.

On line of Michigan Central Railroad. View taken from American side. Bridge consists of one 495-foot cantilever and two 208-foot shore spans, with plate-girder approach. In the background farther down stream, is the single-arch, double-track steel bridge of the Grand Trunk Railway.

PLATE 7—STEEL CONSTRUCTION.

portion were satisfactory. This condition led to the adoption of the names:

Arch-brick, or those forming the top of the oven;

Soft or **salmon brick,** or those named from the lack of color or hardness;

Body brick, or those taken from the middle of the kiln.

In the kilns of modern design, the heat is well distributed throughout the whole mass, and the above terms are not now as appropriate as formerly.

(3) The form and the use for which the brick are intended make the following terms suitable:

Face brick, or those of best appearance as to uniformity of color and dimension, and thus suitable for outside walls. The term is especially used to designate re-pressed brick.

Compass brick, or those having one edge shorter than the other. They are used in building curves on which the length of the brick is a part of the circumference.

Feather-edge brick, those similar to compass brick, but having one edge thinner than the other. They, also, are used in building arches.

Paving brick, or **vitrified blocks,** usually larger than ordinary bricks, used in paving streets.

Sewer brick, or common brick of better grades, so regular in form as to be suitable for building sewers.

Sizes and Weight of Brick. There is no uni-

TABLE XI

Compression Tests of Brick

Dimensions (Inches)			Ultimate Load Pounds	Compression Strength Pounds per Square Inch
Width	Length	Depth		
3.85	3.25	2.35	223,000	7,030
3.95	3.25	2.40	259,000	8,020
3.92	3.90	2.37	343,000	10,700
4.00	3.32	2.35	365,000	11,000
3.95	3.25	2.30	462,000	14,150
3.90	3.20	2.30	453,000	14,150
4.07	3.40	2.55	33,200	2,480
4.05	3.35	2.40	92,800	2,750
4.00	3.00	2.40	125,000	3,980
4.00	3.25	2.40	134,300	4,070
4.00	3.00	2.30	181,500	5,670
3.75	7.60	2.45	233,900	8,390

NOTE—All bricks were imbedded in plaster of Paris before being tested.

form standard of size of brick in this country; but 8¼ by 4 by 2¼ may be said to be the most common dimensions in inches. Face brick are often one-eighth of an inch larger on each dimension. Paving brick are about 3⅛ by 4 by 8¾ inches.

Bricks weigh from 100 pounds to 150 pounds per cubic foot, according to density. Common bricks weigh about 125 pounds per cubic foot.

Specifications for Good Brick. The following specifications for brick are intended as general rather than specific:

(a) A brick should have plane sides, parallel two and two, intersecting in sharp, well-defined lines.

(b) A brick should give a clear ringing sound when struck with another brick or with a

TABLE XII
Transverse Tests of Brick

Dimensions (Inches)		Total Center Load Pounds	Modulus of Rupture Pounds per Square Inch
Depth	Width		
3.97	2.38	4,100	963
3.96	2.38	4,900	1,180
3.85	2.37	5,700	1,450
3.86	2.39	6,100	1,540
3.95	2.48	8,000	1,960
3.90	2.40	8,000	1,970
3.94	2.44	7,600	1,800
4.05	2.45	1,200	292
4.08	2.48	1,900	418
4.05	2.40	2,100	480
4.02	2.40	2,100	480
4.00	2.40	2,000	516
4.00	2.45	2,800	648

NOTE—The bricks were tested on edge with a span of six inches. The load was applied at the middle of the span, and across the upper surface of the brick.

mallet. This will indicate that it is hard, compact, and homogeneous.

(c) A brick should not absorb more than one-tenth of its own weight of water.

(d) The compressive strength of good-quality brick should be not less than 7,000 pounds per square inch, as shown by the tests of half-bricks imbedded in plaster of Paris.

(e) The modulus of rupture of brick should be not less than 1,000 pounds per square inch.

Tests of Brick. Tables XI and XII show the results of some recent tests of shale building brick made in central Illinois. The higher results are from selected hard-burned brick; and the lower, from samples which were soft and under-burned.

BRICKWORK

Brick structures are made by imbedding the brick in mortar made of lime or cement and sand. The mortar is used for two purposes: it serves to cement the bricks together so that they act as a unit, and it also acts as a cushion to distribute the pressure.

If ordinary mortar of lime and sand be used, that will be the weakest part of the brick wall,

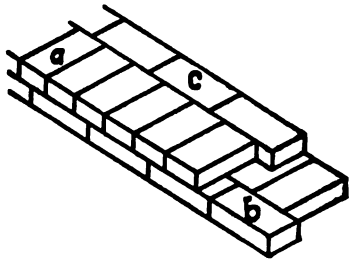


Fig. 46. English Bond.

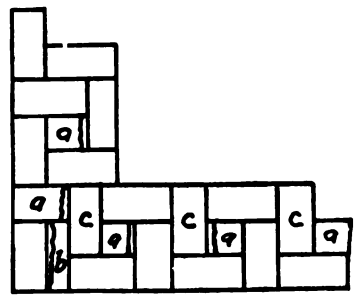


Fig. 47. Plan of Corner Built with Flemish Bond.

and the joints should be kept thin, not over a quarter of an inch in good work. If Portland cement mortar be used, it may be as strong as the brick, but in this case economy usually prompts the builder to make thin joints.

Laying the Brick. Brick are usually quite porous, and, if laid dry, will extract much of the water from the mortar. The effect of this is to weaken the mortar; and so the practice is to wet the brick before they are laid. It is common practice to specify that the brick are to

be laid with a **shove joint.** This means that the mortar is to be spread on first, then the brick is laid lightly in it, and shoved diagonally back from the face and up against the adjacent brick. This manner of laying forces the mortar into the irregularities of the brick, and increases the adhesion between the two substances.

Bond. By this term in brickwork is meant the arrangement of the brick so that each one

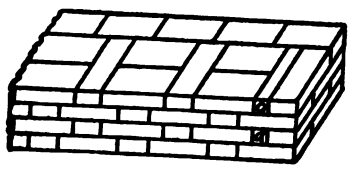


Fig. 48. Flemish Bond.

tends to transfer vertical pressure to two or more below in such a way that no single tier of brick acts alone. This is accomplished by means of **headers** and **stretchers,** as in stone masonry. The two styles of bond in most common use are the English and the Flemish bond.

English Bond. This manner of laying is shown in Fig. 46, where all the bricks in the face of the course **a** are **headers.** In the face of the course below, all are **stretchers.** It is usual to have one row of headers to every half-dozen rows or so of stretchers, although the true bond of this type has them in alternate layers. The arrangement of the bricks in the back, as at **c,** depends upon the thickness of the wall. A piece of brick of less than full section in the face or

in the interior of the wall, used to fill in where a whole brick would be too large, is called a **bat.** As it is not always possible to build a wall of entire bricks, bats are used to make the face appearance uniform. The joints in the header courses must be made thinner than the others to preserve the proper spacing, since there are more of them.

Flemish Bond. This bond makes a very strong wall, and the face presents a particularly pleasing effect. It is considered more difficult

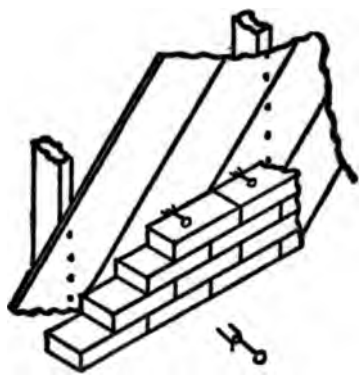


Fig. 49. Method of Bonding Brickwork Veneer to a Wooden Building.

to lay; but it is in common use. Figs. 47 and 48 show a wall laid up in this manner. Here, again, the thickness of the wall and the frequency of the corners determine the arrangement of the bats to make the face regular. Occasionally both sides of the wall are faces on which the appearance must be that of alternate headers and stretchers. Many apparent headers

are only bats, even in walls of good workmanship; and the real headers are not necessarily of full cross-section.

When part of the bricks are darker than others, the dark ones are frequently selected as headers, with pleasing results.

A corner built with Flemish bond to look alike in front and back, is shown in Fig. 47. Bats are shown at points marked **a**. The brick **b** is split to secure the proper spacing in both front and back. The course above that shown is built to break joints with this one. The face of the wall is like that in Fig. 48. This is a thirteen-inch wall in which the narrow brick is shown at **a**.

A method of bonding a veneer of brickwork to a wooden building by means of iron wire hooks, is shown in Fig. 49.

STRENGTH OF BRICK MASONRY

The strength of brick is some indication of the strength of the masonry made from it, but the mortar and the workmanship must also be taken into account in making comparisons.

Table XIII gives results of some recent tests of brick columns made of brick similar to those referred to in Tables XI and XII. The columns were 12½ inches square, and were from 40 to 43 courses of brick high. The letters at the left refer to groups of several tests. Group **H** is of under-burned clay brick. The load in **C** was applied one inch from the center.

TABLE XIII

Summary of Tests of Brick Columns

Average Values

Reference	Characteristics of Columns	Average Unit-Load (Lbs. per Sq. In.)	Ratio of Strength of Column to Strength of Brick	Crushing Strength of 6-in. Mortar Cubes (Lbs. per Sq. In.)	Ratio of Strength of Column to Strength of Cubes
	SHALE BUILDING BRICK				
A	Well laid, 1:3 Portland cement mortar 67 days..	3,965	.31	*2,870	1.17
B	Well laid, 1:3 Portland cement mortar 6 months	3,950	.37	.....	.....
C	Well laid, 1:3 Portland cement mortar, eccentrically loaded, 68 days.....	2,800	.26	.....	.....
D	Poorly laid, 1:3 Portland cement mortar, 67 days..	2,920	.27	*2,870	1.05
E	Well laid, 1:5 Portland cement mortar, 66 days..	2,225	.21	1,710	1.30
F	Well laid, 1:3 Natural cement mortar, 67 days..	1,750	.16	365	5.75
G	Well laid, 1:2 Lime mortar, 66 days.....	1,450	.14	.....	.....
	UNBURNED CLAY BRICK				
H	Well laid, 1:3 Portland cement mortar, 68 days..	1,060	.27	*2,870	.37

*Average value based on 13 tests of 1:3 Portland cement mortar cubes 60 days old.

The results shown in Table XIII indicate that both the strength of the individual bricks and that of the mortar are factors in the strength of the columns. As it is the weakest part that fails, it is evident economy to have the strength of the various parts the same.

The building laws of the city of Chicago provide that "brickwork in walls laid in standard Portland cement mortar shall not be loaded

more than 25,000 pounds per square foot. Brickwork in an ordinary cement mortar shall not be loaded more than 18,000 pounds per square foot. Brickwork in walls laid in lime mortar shall not be loaded more than 13,000 pounds per square foot."

When we consider the results of tests as shown in Table XIII, it is thus seen that a large factor of safety is allowed in practice.

Brickwork Laid in Freezing Weather. When it is necessary to lay bricks in freezing weather, there are several ways to counteract the effect of the cold. It should not, however, be assumed that any of the suggested plans are entirely satisfactory. The brick and sand may be heated before being laid; the space to be occupied by the wall may be enclosed within a temporary structure; or the freezing point of the mortar may be lowered by the addition of salt to the water. This last method should be used only when the brick themselves have been warmed so as to remove ice and frost from the surface. If this is not done, there will not be firm adhesion between the mortar and the brick.

The rule for mixing mortar has already been given in the section on Stone Masonry, but will be repeated here with a little different wording. Dissolve one pound of salt in eighteen gallons of water for working in a temperature at freezing point; and for lower temperatures, use one ounce of salt additional for each degree that the temperature is below freezing. Enough

salt must be used to prevent the mortar from freezing, whatever the temperature may be.

Measurement of Brickwork. It is customary to estimate brickwork by the thousand brick contained in the wall. As many parts are not open to inspection, it is clearly not possible to determine the number by actual count, and recourse must be had to measurement.

The work of building around openings is not nearly so difficult with brick as with stone; and accordingly not so great an allowance for openings need be made in the case of brickwork as in the case of stone masonry.

The actual number of bricks in a cubic foot varies with the thickness of the mortar joints as well as with the size of the brick; but usually the average is between 17½ and 19½. An average of 18 may be taken as very nearly suitable in most cases when the work is continuous, and the joints one-quarter of an inch thick. When there are many openings and small piers, requiring much cutting and consequent waste, it is safe to allow 20 brick per cubic foot for the work.

When asking for bids on work, it is well to specify just how the measurement of the quantity shall be made; and the following rule is to be recommended as fair and tending to prevent controversy:

Divide the total number of superficial feet of wall surface of a given thickness by 160, and

multiply the result by the number of brick widths the wall is thick. The result will be the number of thousand of brick contained.

This rule is based on the fact that a 4-inch wall contains about 1,000 brick to 160 superficial feet, if the joints be ¼ inch thick.

For example, a 12-inch wall 40 feet long and 20 feet high would contain $40 \times 20 \div 160 \times 3 =$ 15 thousand bricks.

While the above rule gives the number of brick to be purchased for the wall, another arbitrary rule for the payment of the masons is sometimes adopted:

Count 7½ bricks for each superficial foot of wall for each half-brick (half-length of brick) thickness of wall.

In the same example as above, this would give $40 \times 20 \times 7\frac{1}{2} \times 3 = 18$ thousand bricks as a basis of payment for the labor.

The **allowance for openings** is not a uniform practice, but may be fairly well established by custom in any given city. These customs must be consulted in letting or computing such work.

When no deduction is made for openings, it is not customary to make extra payment for arches, buttresses, pilasters, and the like.

Amount of Material Required. In estimating the cost of a proposed piece of brickwork, the rule given above determines the number of thousand of brick required. The amount

TABLE XIV

Mixing Table for Mortar for Laying 1,000 Bricks

Mortar	Lime Barrels	Cement Barrels	Sand Cubic Yards
Lime ..... 1 Sand 3	1	.....	0.5
Portland Cement..... 1 Sand 3	.....	1.5	0 5
Natural Cement..... 1 Sand 3	.....	1.5	0.5
Portland Cement..... 1 Lime ..... 1 Sand 8	1	1	0.5

of sand, lime, or cement depends on the richness of the mortar and the thickness of the joints.

Table XIV gives the necessary amount of sand and cement to lay 1,000 bricks with joints about 3/8 inch thick.

Stated otherwise, a cubic yard of brick masonry with 1/2-inch joints requires rather more than 1/3 of a cubic yard of mortar. If the joints are 1/4 inch thick, 1/4 cubic yard of mortar will be sufficient to lay a cubic yard of brickwork. When joints are only 1/8 inch thick, 1/8 cubic yard of mortar will be required to a cubic yard of brickwork.

A barrel of lime will make 2 1/4 barrels (or .3 cubic yard) of lime paste. A barrel of this paste, with 3 barrels of sand, will make 3 barrels of lime mortar. A barrel of unslaked lime is sufficient for 6 3/4 barrels of 1 to 3 mortar.

Data Concerning Brickwork. The weight of brick walls, at 112 pounds per cubic foot, is:

9-inch	wall	.....	84	pounds	per	superficial	foot
13	"	"	121	"	"	"	"
18	"	"	168	"	"	"	"
22	"	"	205	"	"	"	"
26	"	"	243	"	"	"	"

A load of sand or mortar is a cubic yard.

Sand weighs from 80 to 115 pounds per cubic foot, or 1 to 1½ tons per cubic yard.

A cubic yard of mortar requires a cubic yard of sand, and equals 30 hodfuls.

A bricklayer's hod holds 20 bricks.

Masonry with ⅜-inch joints contains 500 bricks per cubic yard. With ⅛-inch joints the number will be 575.

No eight-inch brick wall should be over 14 feet in height.

A mason and his helper should lay 1,000 bricks in seven hours on plain work.

Flemish bond costs more to lay than plain or English bond.

The waste of brick in good material is about 2 per cent for ordinary work; in soft or salmon, as much as 5 per cent.

Walls that are a brick-length thick are often called 9-inch walls.

A 13-inch wall is a brick-length, plus its width, plus the thickness of a mortar joint.

A 17-inch wall has a thickness equal to the length of two bricks, with the mortar joint.

A barrel of Portland cement, nominally 400 pounds, weighs about 380 pounds net.

A barrel of natural cement, nominally 300 pounds, weighs about 265 pounds net.

A barrel of lime weighs about 200 pounds.

Two and a-half bushels equals a barrel.

AN EDUCATION IN STEEL RIVETING, BUILDING UP, AND CONNECTING.

Richmond Viaduct and Terminal Structure, Chesapeake & Ohio Railway, Richmond, Va.

Elements of Steel Construction

RAW MATERIALS

1. **Introduction.** The iron and steel industry is one of our greatest, and yet it is comparatively young. While iron was known to the ancients, it is only within the last forty years that it has been manufactured in large quantities. Indeed, it has been estimated that more iron and steel have been produced within the last fifty years than the entire quantity produced in all the previous centuries.

Two causes can be assigned in explanation of the marvelously rapid expansion of the industry. The first was the invention of the steel-frame fireproof structure, which, in the construction of office and industrial buildings, has largely taken the place of the old-time timber and masonry. This type of building may be said to date from the erection in 1884-5 of the Home Insurance Building, in Chicago, Ill.—the original "sky-scraper"—which was designed by W. L. B. Jenney, a Chicago architect, and which soon attract-

ed a world-wide attention and following that opened up an unlimited field for the application of steel shapes in the erection not only of buildings, but also of bridges and other structures.

The second cause which has played an important part in contributing to the increased demand for structural steel, has been the distinctively modern alliance of steel and concrete in what is known as the reinforced concrete type of construction, dating from about 1895. The increasing structural applications of **reinforced concrete**—or concrete-steel, as it is sometimes known—is one of the most striking features of the present era of industrial development.

In point of capital invested ($1,528,979,076), annual wages paid ($381,875,499), and gross value of yearly product ($803,968,273), the iron and steel industry stands first in rank; while in number of employees (733,968) it is second, that of the manufacture of textiles being ahead*. When it is seen that the number of employees is greater than the combined population of the cities of Cincinnati and Buffalo, is half as great as the population of Chicago, and is greater than the combined number of people in the States of Maine and Nevada, and that if the yearly income from the industry were applied on the national debt it would wipe it out in about thirteen months, we can readily appreciate the magnitude of its importance.

2. **Iron Ore—Production and Classes.** Iron

* 12th Census Reports.

PRATT TRUSS RAILROAD BRIDGE.

Single-track truss in foreground; double-track drawbridge in middle. Note method of carrying telegraph and telephone wires over the bridge.

Courtesy of American Bridge Co.

PENNSYLVANIA RAILROAD BRIDGE OVER FREIGHT YARD AT 52D STREET, PHILADELPHIA, PA.

One single-track through pin-connected truss span, 387 ft. 10 in. center to center of end pins. Weight, 1,800,000 lbs. Built in 1902.

ore occurs in almost every country of the globe. In its production the United States stands first, with 52,955,070 long tons in 1907—or about fifty per cent of the world's output. Germany comes next with an output almost equal to that mined in the State of Minnesota. Great Britain comes after Germany; and then come Italy, France, Russia, and Austria-Hungary, with outputs scarcely equal to or little larger than that of our two greatest mines.

Iron ore is divided into classes according to its chemical composition. The various kinds are: **Magnetite, Brown Hematite** or **Limonite, Red Hematite,** and **Carbonate** or **Siderite.** Magnetite is so called on account of its peculiar property of affecting the magnetic needle. The difference between brown and red hematite is that the "brown" has water occurring in it in chemical combination, while the "red" has not. All iron ore may have water occurring in it in a mechanical way—that is, as dampness—in which case it is heated in large tanks and the water driven off as steam. This is done in order to reduce the weight and therefore the freight charges, and also to reduce the amount of fuel required for turning the ore into cast-iron. Practically ninety per cent of all iron ore used in this country is red hematite.

The United States imports a small amount of iron ore, almost all the imported supply coming from Cuba, which ships us about one-half of its production.

Of the 52,955,070 tons mined in the United States in 1907, 42,245,070, or about eighty per cent, came from the "Lake Superior Region," which is that portion of the States of Michigan, Minnesota, and Wisconsin lying at the western

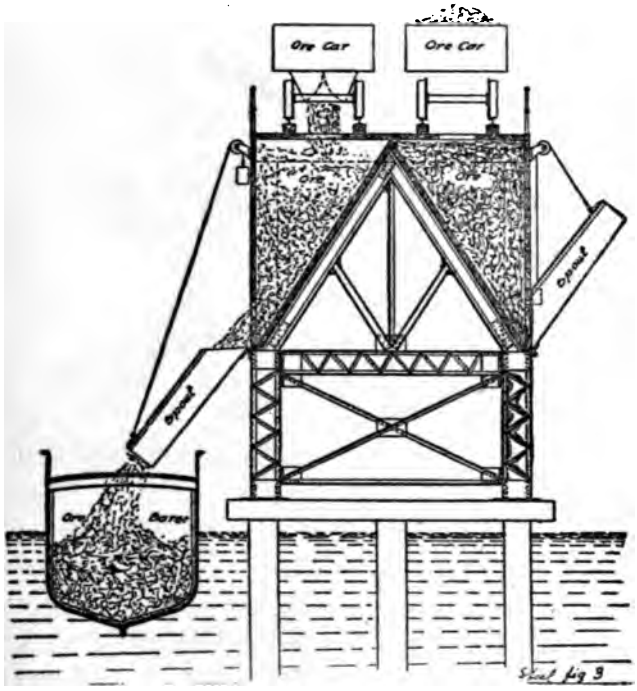


Fig. 1. Section of Ore Dock.

end of Lake Superior. Of this amount, Wisconsin produces about two per cent. Alabama is the greatest iron ore State outside of Michigan and Minnesota, since it produces about one-half of the United States ore which is not produced in the Lake Superior Region.

3. **Occurrence.** The ore may occur at con-

siderable depth below the surface, in which case it is mined by the use of shafts and cross-headings, the method being similar to that employed in coal mining; or it may occur in form like loose clay and in huge pockets, or in heaps or small mountains, at a short distance under the surface, in which case the surface dirt is stripped off by means of steam shovels and the ore is loaded on by the same means. Many of the mines in the Lake Superior Region are of this kind, and the ore is taken out very economically, loaded on cars, and shipped to the ore docks at the lake. Here it is stored in the dock hoppers; and when a boat comes to be loaded, spouts are lowered to the several hatches of the boat, and the coal thus delivered on board. See Fig. 1 and Plate 1. These huge steel barges can carry as much as 12,000 to 13,000 tons at a time, can be loaded at the rate of about 100 tons a minute, and unloaded at the rate of 35 tons in the same period of time.

After the load has been put on board these barges, for only about one per cent of the ore of the Lake Superior mines is carried all the way to the furnaces by rail, they proceed in tow or under their own steam to the port of Gary, Indiana, at the southern end of Lake Michigan, or to one of the ports on Lake Erie, where they are unloaded and their contents either put in large piles or taken directly to the furnaces, a large majority of which are in Pennsylvania.

4. **Ore Ranges.** The ore fields of the Lake

Superior Region are named after the respective ranges in which they occur. Table I gives the names and location of these ranges, as well as their output for 1907, and the approximate date of opening. The approximate cost of a ton of ore delivered is $4.00.

TABLE I

Data of Lake Superior Region

Range	Location	Output, 1907	Date Opened
Marquette	Michigan	4,333,073 tons	1854
Menominee	Michigan and Wisconsin	4,964,728 "	1877
Gogebic	" " "	3,637,907 "	1884
Vermillion	Minnesota	1,635,267 "	1884
Mesabe	"	24,492,949 "	1892
Baraboo	Wisconsin	76,146 "	

CAST IRON

5. **Manufacture.** The amount of pig-iron obtained from a ton of ore depends on the richness of the ore—that is, on the amount of pure iron that it contains. The richness of ore varies considerably, the percentage being low in some of the lean ores of New Jersey, and high in the rich ores of the Lake Superior Region. The latter contain about sixty to sixty-five per cent of iron when the usual ten or twelve per cent of water is driven off.

In order to make cast iron, ore, together with about two-thirds to three-fourths of its weight in coke and a certain proportion of limestone, is dumped in the top of a blast furnace. This furnace should be of the general form and dimensions of Fig. 2. It should be from 65 to 100 feet high. Eighty feet, with an angle of the boshes of

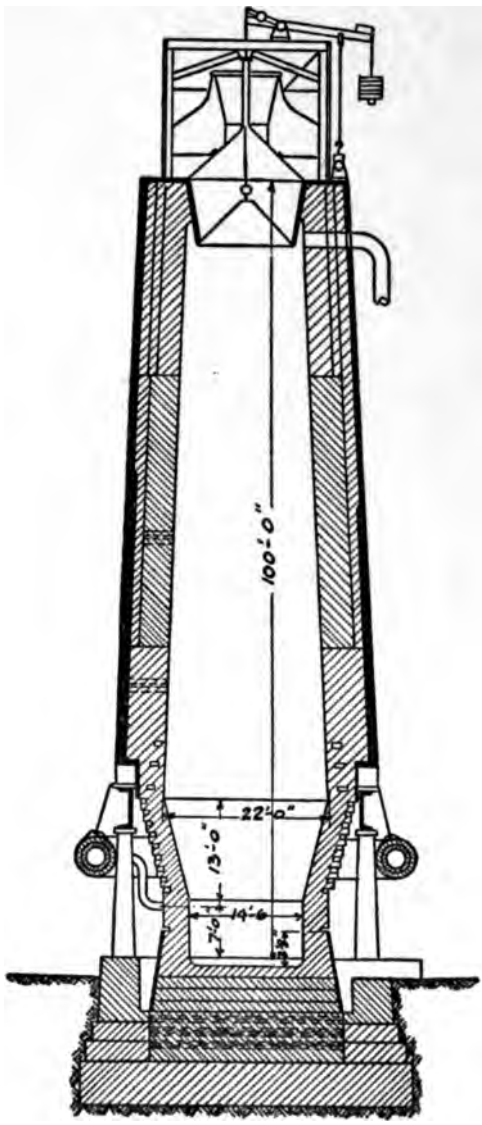


Fig. 2. Section of Blast Furnace.

75 degrees, has been proved about the best size. A wide difference in results is obtained from ores of different percentages of richness and in different districts. American ores are unsurpassed in quality, and a comparison of working conditions in blast-furnace practice shows also that American methods are unequaled in general efficiency.

The fire is in the lower part of the furnace; and as the charge which was dumped in the stack descends to where the fire is burning, changes take place. The charge finally reaches the flames, which are fed by the coke, and they in turn melt the iron in the ore. Many of the impurities in the ore and in the coke—such as phosphorus and sulphur—combine with the limestone, which by this time has been burned to lime, and form what is known as a **slag**. The iron, being the heaviest of the materials in the furnace, sinks to the bottom, and the slag floats on top of it. At certain periods when the iron gets high enough in the hearth, the slag is drawn off from the top, and the iron is drawn out through an opening at the bottom. Fresh charges of ore, fuel, and limestone (which is called a **flux**) are continually dumped in at the top, so as to keep the stack full.

In order to obtain the intense heat necessary, air under considerable pressure is blown in at the bottom of the stack through pipes called **tuyeres** (pronounced tweyers). This air furnishes means for increasing the draft. It passes

up through the furnace to the throat of the furnace, and then, with other gases, passes out through the down-runner pipe which leads to the **hot-blast stoves.**

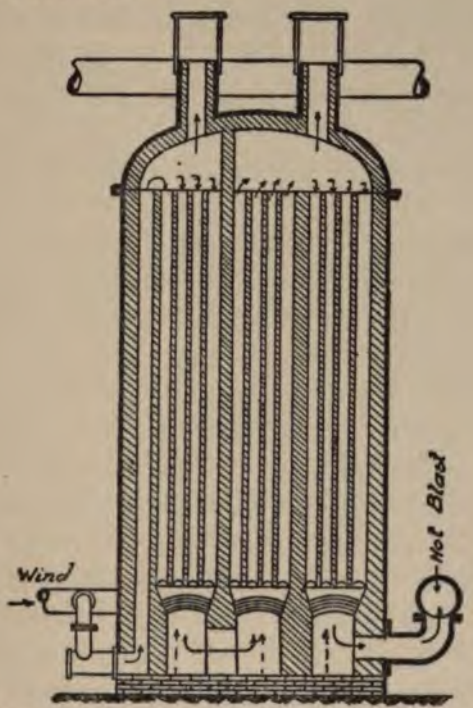


Fig. 3. Interior of Hot-Blast Stove when Blast is Going through It.

These stoves are huge, hollow iron stacks built in pairs and filled with a checkerwork of fine brick. See Fig. 3. The burning gases from the furnaces are led into one of these, and allowed to pass upward to the top, burning all the time. This heats up the bricks to a white heat. The burning gases are then shut off from

this stove, and are turned into the other one of the pair; and the air which was going through the other stove, and which is used for the blast for the blast furnace, is now turned into the stove which has just been heated. The air gets heated by going through this stove, and then it goes into the blast furnace. In this manner, one stove is being heated by the waste gases from the furnace while the other one is heating the blast.

The heating of the blast effects a great economy of fuel in the blast furnace. By these stoves, the air can be heated up to 1,000° F. to 1,200° F. Plate 2 shows blast furnaces and hot-blast stoves. These stoves are sometimes called after the names of their inventors, as "Couper" or "Kennedy-Couper." If the waste gases from the furnaces are more than sufficient to run the stoves, they are conducted to boilers and burned there instead of using coal, or they may be used in the operation of gas engines furnishing power to run electrical generators or other machinery.

6. **Casting.** After the iron has been tapped from the furnace, it is cast into **pigs**, either by being run into sand moulds which form "sand-cast" pigs, or by being run into huge ladles and poured into a machine which casts the pigs. In the latter case the pigs are called "machine-cast" pigs.

In case it is not desirable to cast into pigs, the contents of the ladle are poured into a huge cylindrical furnace which is on rollers. This

mixer, as it is called, holds many ladles full, and by rocking to and fro mixes the different charges so that the total mass is homogeneous. Iron taken from this mixer is used in making steel. Sometimes the mixer is not used, the ladle taking its charge directly to the steel furnaces.

7. **Grading.** Cast iron consists chemically of from 2 to 4 per cent of graphite or uncombined carbon, and from 3 to 0.02 per cent of chemically combined carbon and other chemicals. "Uncombined carbon" means carbon that is mixed in with the iron somewhat in the same manner as if we should make a brick and mix in a lot of sawdust. The combined carbon is fixed in the iron chemically, about like the sand, which, when the brick is burned, turns into a glaze or some other form. The amount of combined and uncombined carbon influences the physical properties of cast iron, as also do the other chemicals.

The grading of cast iron is done according to its hardness, and this depends upon the relative amount of combined and uncombined carbon. Table II, taken from the "Institute Journal" for 1891 (Vol. II, p. 245), gives the grades, and their composition in per cent, as determined from a series of tests on some iron from Alabama. The softest is the "No. 1 Foundry," and the hardness increases until "White," the hardest, is reached.

It must not be supposed that the analysis shown in Table II is always the same for any particular grade, but it will be very closely the

TABLE II

Grading and Analysis of Cast Iron

Hardness Rank	Grade	Percentages of					
		Combined Carbon	Uncombined Carbon	Silicon	Sulphur	Phosphorus	Manganese
1	No. 1 Foundry	3.49	0.07	3.15	0.005	0.68	0.25
2	No. 2 Foundry	3.55	0.07	2.40	0.024	0.68	0.22
3	No. 3 Foundry	3.48	0.10	2.20	0.025	0.64	0.21
4	No. 1 Soft	3.53	0.03	3.75	0.005	0.68	0.27
5	No. 2 Soft	3.48	0.03	3.50	0.004	0.68	0.26
6	Silver Gray	3.13	0.02	5.50	trace	0.68	0.25
7	Gray Forge	3.00	0.57	1.50	0.060	0.64	0.19
8	Mottled	2.11	1.22	1.35	0.125	0.64	0.14
9	White	0.10	2.92	0.95	0.300	0.64	0.10

same. Sulphur tends to cause brittleness; phosphorus increases fluidity, but decreases strength; manganese causes hardness; and certain percentages of silicon make iron soft, while others make it weak and hard. Probably no other constituent has so much effect as silicon on the physical properties of iron; and by varying its percentage, cast iron for different purposes is made.

8. **Output of Cast Iron.** As in the case of ore, the United States leads all other countries in volume of production of cast iron, with a total of 25,781,361 tons in 1907, out of the world's total of 60,680,014 tons. This is 42.4 per cent. Of this amount, the greater part was produced by the following states: Pennsylvania (44 per cent), Ohio (20 per cent), and Illinois (10 per cent). Alabama and New York come a close fourth with 6.4 per cent each.

WROUGHT IRON

9. **Manufacture.** About 500 pounds of cast iron is put into the hearth of a puddling or reverberatory furnace (Fig. 4), and a fire built in the furnace. The heat melts the iron, the impurities form a slag with the iron oxide lining of the hearth, and the carbon burns out. The purer the iron, the higher the heat that is required to melt it; and therefore, as it becomes

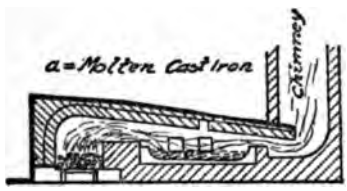


Fig. 4. Section of Reverberatory or Puddling Furnace.

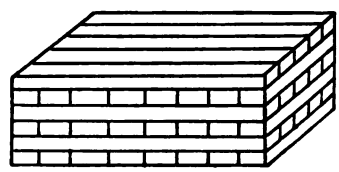


Fig. 5. Pile of Muck Bars.

purer, the heat of the furnace being insufficient, the iron becomes pasty or "comes to nature," as the puddlers call it. The puddler, by means of an iron bar or poker, collects the pasty mass into several balls called **puddle balls.** When these are of the right consistency, he takes them out and places them in a **squeezer.**

The squeezer is a cam-like machine which squeezes most of the slag out of the ball and welds the entire mass into a homogeneous whole. This mass is then put through rolls, and comes out in bars which contain from one to two per cent of slag. These bars are called **muck bars.** The muck bars are then cut in strips and piled

up in piles (Fig. 5), which are tied with wire, raised to a welding heat, and again rolled into rods, bars, or other shapes. The effect of this last rolling is to squeeze out more slag; and the piling helps to form a product in which all the fibers do not run the same way, as was the case in the muck bar. It takes about two hours to run a charge and recharge the furnace.

10. **Characteristics.** Wrought iron is different from cast iron, in that it can be hammered into almost any shape, can be bent double when cold, and can be welded. Wrought iron may be defined as pig iron with its impurities, all of the graphitic or uncombined carbon, and almost all of the combined carbon, burnt out. Silicon, carbon, manganese, phosphorus, and sulphur occur more or less in all ore, iron, and steel, and in fact in most metals. They are called **metalloids.** During the process of puddling they are largely eliminated, so that wrought iron is nearly pure iron.

11. **Effect of Metalloids.** Too much carbon or sulphur will cause the iron to be "red short." This condition causes it to work poorly in the rolls; and the edges of the piece are rough—somewhat the same result as if one tried to work clay that was too dry. Phosphorus is bad, as it tends to lessen the welding properties of the metal.

It has been claimed that the small amount of slag that is left in the iron, usually less than 2 per cent, increases its welding properties and

strength, but tests do not seem to substantiate this.

12. **Output.** Of all cast iron made in the United States, only about three per cent is made into wrought iron. This would make the amount of wrought iron produced in 1907 about 770,000 tons. This amount is probably an overestimate rather than an underestimate. Far more than this amount of wrought iron, however, is used in this country—probably as much as 2,500,000 tons. The difference between the amount made and that used is explained by large imports, and also by the fact that considerable quantities of new wrought iron are made from wrought iron scrap. The best wrought iron comes from Sweden.

STEEL

13. **Crucible Steel.** This is the finest quality of steel. It is made in three ways—namely, by packing pure soft wrought iron in fine charcoal, and raising to a yellow heat; by putting charcoal and crude bar iron in a sealed crucible and heating to a melting heat; or by melting wrought iron in a sealed crucible with a proper proportion of pig. The aim of all methods is the same—that is, to get a malleable metal containing from 0.60 to 1.40 per cent of carbon, and practically free from sulphur and phosphorus. It is to be noted that all these methods take wrought iron, a metal practically free from carbon, and add carbon to the desired extent.

With the first method, the carbon from the charcoal soaks in at the rate of one-eighth inch each 24 hours. With the second, the absorption of carbon is very rapid on account of the fluid condition of the iron. With the third, also, the process is rapid, the pig distributing its carbon to the mixer so as to bring the total to the required result. The second method is the universal practice in America.

After the mixture is put in the cold crucible and sealed, the crucible is put in a gas furnace and kept at the proper heat until the steel is made. The crucibles are then taken out and the material poured into ingots. From 100 to 200 pounds of steel are made from each crucible. The ingots are then rolled into bars and plates, and they are put on the market.

Crucible steel is used to make cutlery and engraving tools, and parts of machines which perform excessive work. Not much is made, and the annual output has increased less than 20 per cent in the last ten years, while that of other steel has increased over 100 per cent. There were 118,000 tons made in 1906. Plate 3 shows men casting a 55-ton ingot of crucible steel. This is at one of the great steel works of Germany.

14. **Bessemer Steel.** In the Bessemer process, from ten to twelve tons of molten cast iron is poured either directly from the blast furnace or from the "mixer," into a pear-shaped **Bessemer converter,** which is tipped over to receive it. The converter is shown in cross-section in Fig.

6. The trunnions are hollow, and the air from the blast enters by these, being led down to the bottom, where it blows up through the holes there, and then through the charge. The blast is turned on, and the converter tipped into a

Fig. 6. Section of Bessemer Converter in Action.

vertical position. The oxygen of the air combines with the various metalloids, and in from nine to ten minutes nearly all the carbon, manganese, and silicon are burnt out and form slag on top. The phosphorus and sulphur remain the same, and for this reason ores low in phosphorus and sulphur must be used in making cast iron for Bessemer steel. When the carbon is about all burned out, the flame from the converter drops to about one-fourth what it is at

full blast, and it is then time to **recarburize** the charge. As it stands at the end of the carbon flame, it consists of a bath of molten iron containing about 0.4 per cent of carbon, and with a lot of iron oxide floating on top. A carefully calculated amount of manganese and silicon, and enough carbon to bring it up to the desired percentage of that ingredient, are added in the form of **Spiegeleissen,** a mineral which contains the above metalloids. The manganese returns some of the iron of the oxide and clears the charge of loose oxygen, while the silicon prevents the occurrence of bubbles and blow-holes when the material is poured.

The lower the percentage of carbon, all other things being equal, the lower the tensile strength and the softer the steel. The amount of carbon added is therefore a necessity, since, as the bath stands at the end of the blow, it would produce a steel of about the consistency and strength of wrought iron; and, indeed, considerable quantities of steel are so made, being used for many purposes where wrought iron was formerly used.

The material is now poured into a ladle, and from that into ingot moulds. For a description of ingots, see Article 16.

Bessemer steel is not so homogeneous nor so reliable as crucible or open-hearth steel (see next article), on account of the fact that in replacing the carbon which has been burned out, it must be done by a small charge, and there is

Courtesy of American Bridge Co.

PENNSYLVANIA RAILROAD BRIDGE AT PORT PERRY, PA.

Five double-track plate-girder spans, five double-track lattice truss spans, and one double-track through pin-connected span. Weight, 6,361,000 lbs.

Courtesy of American Bridge Co.

UNION RAILROAD BRIDGE OVER MONONGAHELA RIVER AT RANKIN, PA.

Total length of structure, 4,779 ft. Length of channel span, 596 ft. 8 in. Weight of steel used, 2,640,000 lbs.

PLATE 11—STEEL CONSTRUCTION.

Courtesy of American Bridge Co.

ROLLING-LIFT BRIDGE OVER NORTH BRANCH OF CHICAGO RIVER, CLYBOURNE PLACE, CHICAGO, ILL.

One three-truss lattice span 128 ft. long. Weight, 1,830,300 lbs.

PLATE 12—STEEL CONSTRUCTION.

therefore the liability that the carbon will not be uniformly distributed throughout the mass of ten or twelve tons. Bessemer steel is therefore not specified for use in bridges, buildings, and structures in general; but for rails, farm machinery, wire, and nails, it is in general use.

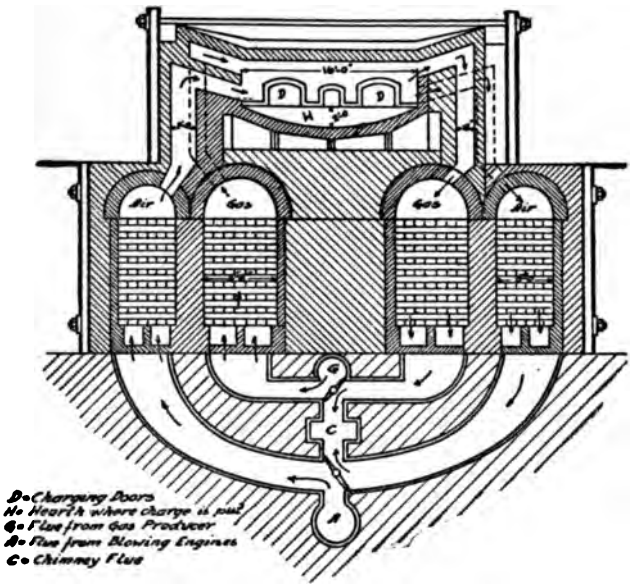


Fig. 7. Section of Open-Hearth Furnace, with Regenerative Furnace, and Diagram of Air and Gas Valves.

15. **Open-Hearth Steel.** In this case the furnace used is as shown in Fig. 7. The depth of molten metal should be 18 to 24 inches in a furnace of from 30 to 50 tons capacity. The charge consists either of cast iron (melted in some cases) or of cast iron and scrap. The stock must be low in phosphorus and sulphur, or the steel will not be of use, and also the lining of

the furnace will be destroyed by the phosphorus attacking it. But, in case low phosphorus and sulphur ores are not available, the furnace may be lined with lime, magnesite, dolomite, or some other like material, and a quantity of limestone or limestone and ore added to the charge. In this latter case, the phosphorus will not ruin the lining, and it will be taken from the metal by the lime and ore and transferred to the slag. The sulphur will be eliminated in part by burning out; and most of the remainder combines with certain materials which are added during the heat, and is carried off in the slag.

The charge is melted by the burning of either natural or artificial gas (producer gas) and air forced in under pressure. These enter the furnace on opposite sides of the hearth, join, burn, and beat down on the charge, thus melting it and keeping it melted.

The waste gases then pass out and through a brick checkerwork similar to that in the hot-blast stoves, and then up the stack. When the checkerwork is hot in a pair of these chambers, the valves are reversed; and the gas and air, before burning in the furnace, are heated by being forced through these chambers, the burning gases from the furnace in the meanwhile passing through the checkerwork in the chambers through which the gas and air first were forced. When one set of chambers becomes cooled, the gas and air are forced through the other, and in this manner the gas and air are

heated by one set of chambers while the other set is being heated by the waste gases. And so the operation goes on. Every little while, test bars are made from metal taken out of the bath, and tests made to determine the amount of carbon present. When the carbon has reached the

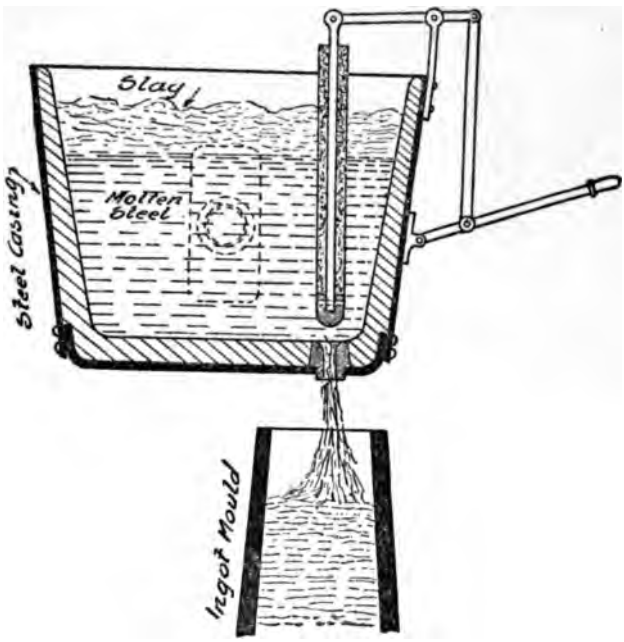


Fig. 8. Section of Valvular Ladle Showing Metal being Poured into Ingot Mould.

percentage desired for the finished product, the **recarburizer** is added. In this case the addition of the "recarburizer" is not, as in the case of the Bessemer process, to increase the carbon, but to remove impurities.

From four to ten hours are required to run a

charge. The time depends upon the amount of carbon to be burnt out. The mixture of scrap and ore with cast iron is usually so proportioned that the bath has from 0.75 to 1.00 per cent of carbon at the start. If a 0.43 per cent carbon steel is required, it will take less time to run the heat than if a 0.23 per cent carbon is required. When the metal is ready, it is run into ladles and poured into the ingot moulds.

It is evident that open-hearth steel is more homogeneous than Bessemer, since the blast can be stopped when the carbon has been burnt down to the required percentage. The mixing has been done by nature, and is therefore evenly done. In the Bessemer process, nearly all carbon is first burnt out, and then enough added to bring the carbon content up to the required percentage.

16. **The Ingot.** In order to prevent the slag from getting into the ingot, the metal is poured from a valvular ladle, Fig. 8. The ingots may be almost any size. The ordinary size for rails is about 63 inches high and 14 inches square, weighing about 2,000 pounds; but from this size the ingots run up to 24 by 26 inches and 5 tons in weight, or even larger, as the case demands (see Fig. 9).

In cooling, the ingot is apt to assume a form like that shown in Fig. 9; and therefore, during cooling, the moulds are covered up with sand or an iron cap. Before the ingots have cooled to anything less than a red heat, they are stripped

of their moulds and placed in **soaking pits.** These are gas furnaces which heat the ingot evenly throughout to the desired temperature for working. The ingot is then taken out and **cropped**—that is, the ends cut off in order to remove any injurious matter. The ingot may then be rolled down and cut into smaller pieces called

Fig. 9. Section Showing Piping of Ingot in Mould.

billets, which are sold to be remelted or heated and have steel articles made from them, or it may immediately be put through the rolls (see Plate 4) and formed into bars, plates, rails, rods, or any of the various shapes for structural use.

17. **Output of Steel.** In 1903 the United States produced over 45 per cent of the world's output of steel, Germany and Great Britain coming next with 24 and 12 per cent respectively.

The percentage has probably not since changed to any great extent. Table III gives the production of Bessemer and open-hearth steel in the three most productive States, as well as in the United States, for the year 1907.

TABLE III

Production of Steel, 1907

Kind of Steel	United States (Tons)	Pennsylvania (Tons)	Ohio (Tons)	Illinois (Tons)
Bessemer	11,667,549	4,851,841	3,636,679	1,723,073
Open-hearth	11,549,088	7,867,705	819,642	1,013,251
Total	23,216,637	12,219,546	4,456,321	2,736,324

It will be noticed that the three particular States mentioned in Table III produce all but about 16 per cent of the output. A noticeable feature of this table is that the output of open-hearth steel is quite up to that of Bessemer. In 1901 the total output was: Bessemer, 8,713,302 tons; open-hearth 4,656,309 tons. On account of the fact that Bessemer steel is not so homogeneous as open-hearth, consumers are using open-hearth in many cases where they had previously used the Bessemer product. Then again, to make Bessemer steel requires an ore low in sulphur and phosphorus, and this is expensive as these ores are becoming scarce. It is safe to predict that the output of open-hearth will always be greater than that of Bessemer and will continue to increase. Also, it seems safe to predict that in the future the production of Bessemer steel will decrease.

STEEL SHAPES

18. **Manufacture of Steel Shapes.** When the ingot is taken from the soaking pit and "cropped," it is placed on the table of the rolls, or **blooming table.** This table consists of a number of cylinders placed close together and made tc revolve either forward or backward as desired. The table moves the ingot to the rolls, where it is rolled through an opening but slightly smaller than the ingot itself. The ingot comes out on the other side longer than it was before, and of uniform section throughout its entire length. Here it is automatically moved to one side, and, the cylinders of the rolling table being put in motion, the steel goes to another set of rolls and is again reduced in sectional dimensions. The first few passes generally reduce the ingot to a rectangular or square section. The steel is then moved sidewise to the different roll openings in turn. Each of these shapes the steel into something approaching its final shape; and at the last pass, what was an ingot only 63 inches long becomes a rail, a plate, or some structural shape 20 to 120 feet or more in length. In cases where the number of passes is great, instead of two rolls, three or more are used. The "blooming table" is so arranged in this case that it may be raised or lowered, thus carrying the steel to the upper set after it has passed through the lower one. Plate 5 (upper figure) illustrates the

rolls in a 16-inch "three-high" mill; and in Plate 4 is seen a blooming table.

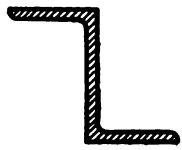
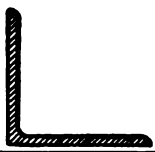
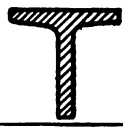
The completed shape is now still at a welding heat. It is run to the **cold saws**, and cut to the required length. This length must be considerably greater than the length required when the piece is cold. As a usual thing, for each ten feet in length of the cold piece, the piece must be cut four inches longer when hot. This of course depends upon the temperature of the steel as it comes from the rolls. The piece is then put on the cooling table and allowed to cool, and is afterwards straightened.

19. **Production of Steel Shapes.** In 1905 there were 5,253,816 tons of structural shapes rolled in the United States. This includes plates and bars and rods, and constitutes about 26 per cent of all the Bessemer and open-hearth steel made in that year. Of this amount, by far the greater part was open-hearth steel.

20. **Steel-Makers' Handbooks.** In order to work in structural steel or to study structural engineering, it is necessary to have a handbook of at least one of the steel companies. Structural steel workers and students can usually obtain these handbooks from the head offices of the companies for 50 cents a copy; to others, the price is $2.00. The handbooks of the Carnegie Steel Company, the Cambria Steel Company, the Bethlehem Steel Company, and others, are equally good. Throughout this text the Carnegie handbook (edition of 1903) will be re-

TABLE IV

Classification of Structural Steel Shapes

Form	Name	Form	Name
	Round (Rod)		Angle (Unequal-Legged)
	Square (Rod)		Channel
	Flat		I-Beam
	Plate		Z-Bar or Zee
	Angle (Equal-Legged)		Tee

ferred to. The number of the page or pages, for example, will be signified thus: (C 45), (C 183-187), or (C 189 and 200), which means that the matter referred to will be found on page 45, or pages 183 to 187, or on page 189 and page 200.

No student of steel construction should be without one or more of these handbooks.

21. **Classes of Shapes.** Shapes for structural work are classified by names characteristic of their form. In Table IV the shapes commonly used in structural work are listed, with a sectional view of each.

Rounds. The circular form of the rounds is obtained by turning them half-way around each time that they are passed through the rolls. They are made from ¼ inch to 6¾ inches in diameter (C 27). They are used in tension—that is, to withstand a pull. The connection at the ends is made by means of a washer and nut. In case the area of the cross-section is required, or the circumference (distance around the outside), it can be found (C 261-266). The large bars listed in (C 263-266) are not, as a general thing, used for anything except to cut in short lengths, turn on a lathe, and use as pins to connect members or as rollers for different purposes.

Squares. These are listed (C 27), and their areas and weights may be found (C 261-266). The larger squares listed (C 263-266) are seldom obtainable except by special order. Their use is confined to supports for bridge ends, and for other similar uses.

Flats. Flats are distinguished from plates, chiefly from the fact that they can be obtained quite thick in proportion to their width, and their width varies by ⅛ of an inch. It is not

advisable to use flats of widths other than 1, 1½, 2, 2½, 3, 3½, 4, 4½, 5, 5½, 6, 7, and 8 inches, as the other widths are seldom kept in stock and have to be made specially to order.

Flats are seldom used separately, except to take tension, and then they are riveted at their ends. It is a common and almost universal use to use flats for tension by first making them into eye-bars. This will be discussed later on.

Plates. Plates are of two kinds—**sheared** and **universal.** This signifies the manner in which they were manufactured. When the ingot is put through the rolls, it comes out from the final pass as a long, flat plate with irregular

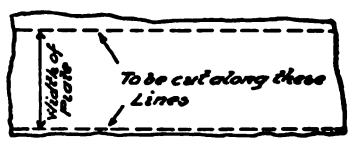


Fig. 10. Sheared Plate Marked for Shearing after it Comes from Rolls.

edges somewhat as indicated in Fig. 10. Straight lines at a distance apart equal to the width of the required plate, are now laid out, and the plate is then taken to the shearing machine and these unequal edges cut off. Sheared plates can always be distinguished by their edges. For sizes and lengths, see (C 30) upper table. At the top of the columns in the table is given a width, and in the next column another width, which differs from the former by a certain number of inches. Suppose, for example, that it

is desired to find the length that a 93-inch plate ¼ inch thick could be rolled. It would, according to the second note at the bottom, be the same as that for the next widest plate given in the table. The next plate is 96-inch, and the length for that is 130 inches. Therefore the greatest length for a 93-inch plate is 130 inches. If it is desired to have a plate shorter than the lengths given, order it, and the mill people will cut off as much as you want. These plates can be obtained in widths of 48, 49, 50, 51, 52, 53, 54, 55, 56, 57, and so on up to 120 inches, but they cannot be had in widths of a fraction of an inch. If a plate 51¼ inches is desired, a 52-inch plate must be ordered, and ¾ inch cut off one side.

Universal plates are rolled by four or by six rolls, one pair horizontal and one pair vertical, or one pair horizontal and two pair vertical. Plate 5 (lower figure) illustrates a universal plate roll. The pairs of rolls can be brought as close together or kept as far apart as desired. In this process the plate is rolled to the desired width as well as thickness, and therefore no shearing is necessary. One great advantage of these plates is that the edge, not being cut, has the same "skin" or surface as the sides of the plate. It will, as a result, take paint much better and will not rust so readily.

The sizes and lengths of universal plates are given in (C 30), lower table. As in the case of sheared plates, these plates may be purchased in any inch in width from smallest to the largest,

but not in any fraction of an inch except in the case of large orders.

Universal plates should be used when possible. They are not used by themselves except to make eye-bars, but they are much used in making **built-up members**, which comprise a great variety of structural members, consisting of several shapes riveted together.

All plates should be listed by writing first the number of plates of each size; then the width in inches; then the thickness in inches; and finally the length in feet and inches. For example, if we wanted a plate 18 inches wide, $\frac{3}{8}$ inch thick, and 12 feet $5\frac{1}{2}$ inches long, we should specify it thus: 1 Pl. $18'' \times \frac{3}{8}'' \times 12'\text{-}5\frac{1}{2}''$; or, if the length is less than a foot, 1 Pl. $7'' \times \frac{1}{2}'' \times 0'\text{-}8\frac{1}{4}''$. If two or more plates are to be listed, we should write: 2 Pls. $17'' \times \frac{1}{2}'' \times 5'\text{-}6''$, or 7 Pls. $3'' \times \frac{5}{8}'' \times 18'\text{-}6\frac{1}{8}''$.

Angles. The angle is the most useful of structural shapes. There is scarcely any built-up member of any size which does not of necessity require angles in its construction; and they are employed either singly or in a pair or pairs to make many smaller members. The angle may be used to take either tension or compression.

Angles are so called because of their shape, the two legs meeting to form an angle; and they are made **with equal legs** (C 13-14) and **with unequal legs** (C 14-15). Special angles—**square root angles**, so called because their angles are

all squared—are given (C 16-17), but are very rarely used.

In the tables (C 109-119) are given various useful data relating to angles with unequal and with equal legs. For the present the first five columns will be noted. In column 1 is given the mark by which the rolls can be picked out when it is desired to roll that section. Those sections which have a star (*) are special angles. They are not square root angles, but the star indicates that they are not kept in stock, and are not rolled except in large tonnages, unless a high price is charged for them. They should not be used in the design of structures. Some one of the other angles will do.

The size, thickness, weight per foot, and the number of square inches in the section, are given in the next four columns. Do not use an angle thicker than 13/16 of an inch if you can well avoid it. This is about as thick as can safely be punched for rivets. For thicknesses greater than this, the holes must be bored, and this is very expensive. It is, of course, possible to punch material thicker than 13/16 inch; but the thickness of the plate in such cases is too great compared with the diameter of the punch (which is seldom more than 13/16 inch), and the punch is likely to break off in the hole. It will be cheaper to use a larger and thinner angle, even if the area is somewhat greater than that required.

In listing angles the following method is in

common use: The large leg (if of unequal legs) should be stated first, the small leg next, the thickness next, and finally the length in feet and inches. The number of the angles required is placed first. For example: 1 ∟ 6″×4″×⅜″ ×17′-6¼″; or 2 ∟'s 4″×4″×¾″×7′-8⅞″.

Members composed of angles are either riveted directly to other members, or are riveted to a plate which is connected to the other member or members by rivets or a plate.

Channels. Channels are used mostly in pairs for compression members—that is, members taking a thrust or push. They are sometimes used as tension members; but, on account of the thinness of their webs, the end connections are expensive because of the large number of rivets necessary. Channels are much used as beams. In (C 101 and 102) is a table containing a large amount of useful data relative to channels.

On account of the many dimensions of a channel, it is simpler to designate them by their depth only, and then give their weight per linear foot, and finally their length. Thus: 1 [9″× 13.25 lbs.—17′-6″; or 2 ['s 12″×20.50 lbs.— 7′-4½″. These signify, in the first case, one channel 9 inches deep, weighing 13.25 pounds per linear foot, and 17 feet 6 inches long; and, in the second case, two channels, each 12 inches deep, weighing 20.5 pounds per linear foot, and 7 feet 4½ inches long.

Those channels whose weights are given in

heavy-faced type (like the 8″×11.25 lbs., 9″×13.25 lbs., 10″×15.00 lbs., etc.) are called **standard sizes.** Formerly it was necessary to avoid using any others but these; but now most of the large steel companies keep all weights in stock, and any weight can be obtained easily. In columns 5 and 6 are given the dimensions of web and flanges; (C 6 and 7) give the sections; while (C 183-186) give valuable information as to dimensions and riveting in flange and in the web.

I-Beams. The shapes known as I-beams—so called from the resemblance of their section to the letter I—are seldom used in any other manner than as beams; see (C 1-5). In some cases, particularly in building members for cranes, they are used to help make compression members. They are much used in building construction, to form what are known as **box girders**; see (C 152-156). An I-beam should be listed thus: 1 I 24″×80 lbs.—3′-4″, which means, one I-beam, height 24 inches, weight 80 pounds per linear foot, 3 feet 4 inches long.

The tables (C 97-100) are for I-beams. They give data for I-beams practically the same as the tables mentioned above do for channels; and the same remarks regarding the tables for channels apply to these for I-beams. On (C 175-182) are given tables which contain information regarding the method of fastening in walls, and the position of rivets when such a connection is to be used.

Courtesy of American Bridge Co.

SCHERZER ROLLING-LIFT BRIDGE OVER CUYAHOGA RIVER, CLEVELAND, OHIO.

On line of Newburgh & South Shore Railway. Weight, 1,510,000 lbs.

Courtesy of American Bridge Co.

STRAUSS BASCULE BRIDGE AT CLEVELAND, OHIO.

On line of Wabash Railroad. Weight, 1,012,400 lbs.

PLATE 14—STEEL CONSTRUCTION.

Courtesy of American Bridge Co.

PAGE BASCULE BRIDGE OVER CHICAGO RIVER AT CHICAGO, ILL.

On line of Chicago & Alton Railroad. Weight, 2,273,000 lbs.

PLATE 15—STEEL CONSTRUCTION.

Z-Bars. Information regarding Z-bars (or Zees, as they are sometimes called) is given on (C 10, 11), (C 103 and 104), and (C 189-190). Z-bars are seldom used for anything except columns. The excellent manner in which they can be combined with a plate to form an economical section for compression, and the many faces they have to which connections can be easily made, make them general favorites for columns. Fig. 11 shows how these can be put together.

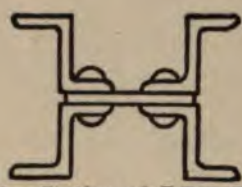


Fig. 11. Section of Z-Bar Column.

Other Shapes. Several other shapes besides the above-mentioned are rolled. These are the **Tees** (see Table IV), the **half-rounds** and **ovals,** and the special kinds of channels, Z-bars, and plates. They are, however, comparatively seldom used in structural work.

RIVETS AND RIVETING

22. Rivets are made in different sizes and lengths. The head and diameter are the same for any given size, but the length will vary as the case requires. When the size of a rivet is spoken of, the diameter is usually meant.

(C 191) gives a table of sizes of rivet-heads,

showing also the clearances necessary for machine driving.

Rivets are made by heating long rods of the required diameter in a small furnace until they attain a welding heat for a distance of about one foot from the end. The operator then takes out the bar, and sticks the end in the rivet machine, which has previously been set so as to make rivets of the required length. The machine grips the rod, forms the head by jamming together a portion of the rod, and cuts the rivet

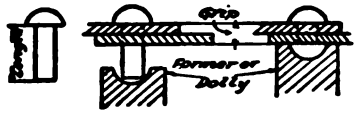


Fig. 12. Length and Formation of Head.

off to the proper length. The rivet drops into a receptacle; the machine releases the rod, and the operator shoves it further in, and another rivet is made. This operation is kept up until all the heated part of the rod has been made into rivets. The operator then returns the remainder of the rod to the fire, to get it heated up again, takes out another, and makes rivets out of its heated portion. This action is repeated until all the rivets required are made. If a rod gets so short that the end not in the fire becomes too hot to handle, it is put in the furnace and handled with tongs. In this way the entire rod is used up. As one rod is used, others are put in, and thus the process continues as long as desired.

Rivets were formerly made from a pure grade of wrought iron; but now it is the general practice to make them of soft steel, a material much like wrought iron in physical properties.

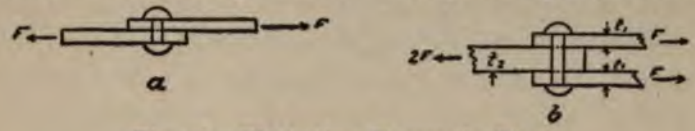


Fig. 13. Illustrating Stresses in Rivets.

The length of the rivet must be such that it will go entirely through the material which it is to hold together, and in addition extend beyond far enough to afford material for forming the other head. Fig. 12 shows the length, grip, and formation of a second head.

Stresses in Rivets. The stresses which come on a rivet are: **Shear,** which is the tendency to be cut across its section; the **bearing**

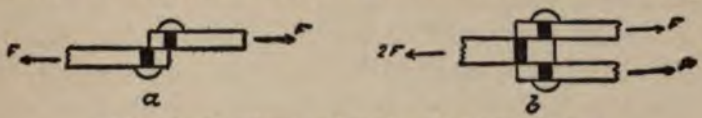


Fig. 14. Illustrating Single and Double Shear.

on the plates it connects; and the **tension** on the heads, due to contraction. Referring to Fig. 13 (**a** and **b**), it will be seen that if the joints are like (**a**) and the two plates which are pulled in opposite directions are so strong that the rivet will not pull out of them, then it will be cut in two at the point between the plates, and will become as Fig. 14 (**a**), being sheared off in a manner somewhat the same as if it were cut by a

pair of shears. When the conditions are such as shown in Fig. 13 (a), the rivet is said to be **in single shear.**

In order to find out how many pounds a rivet should stand in single shear, multiply the area of the rivet by the allowed unit shearing stress. In (C 195), fourth column, are given, in the several tables, the number of pounds a rivet should stand if the allowable stress on one square inch of metal is as stated. Thus in the first table the allowable unit-stress is 6,000 pounds; and if the rivet is $\frac{7}{8}''$ in diameter, we look in the first column and find $\frac{7}{8}$, and then to the right on the same line in column 4 is found 3,610, which is the value sought. That is, in a rivet joint 3,610 pounds can be allowed for each rivet driven. This allowable stress is quite low, and will perhaps be used in railroad bridge design.

For building design, the last table should be used; and here in the fourth column, opposite $\frac{7}{8}$, is found 7,220, which is the number of pounds one rivet should stand when the unit shearing stress is 12,000 pounds. The other tables give values for unit-stresses of 7,500 and 10,000 pounds.

In case the joint is as in Fig. 13 (b), the rivet will, providing the plates are strong enough to keep it from pulling out, fail by being cut off at each of the joints between the middle and side plates, Fig. 14 (b). Since just twice as much will be cut as in the case of Fig. 13 (a), the rivet is said to be in **double shear.**

Since, in a riveted joint, we do not wish the rivet to fail, we must select some unit-stress less than enough to destroy it. Such stresses are given (C 195), being 6,000, 7,500, 10,000, and 12,000 lbs. per square inch.

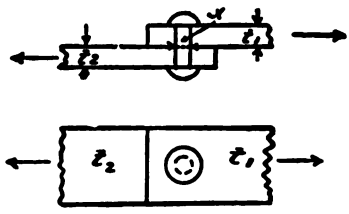


Fig. 15. Illustrating Bearing Stress in a Rivet.

In order to get the number of pounds which should be allowed on one rivet in double shear, the value for single shear is found, and then doubled. For example, to find the value of a ¾-inch rivet in double shear, under an allowed unit-stress of 10,000 pounds, the value for single shear (4th column on same line as ¾ in the 1st column) is 4,420 pounds; and therefore the

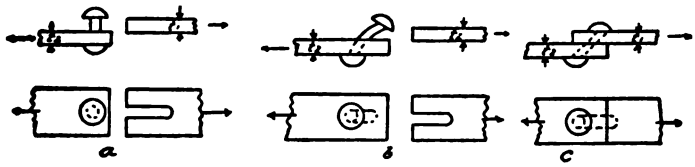


Fig. 16. Illustrating Failure of Joints by Bearing.

value in double shear is 4,420×2=8,840 pounds.

It is very evident that, where possible, rivets in double shear are the more economical, since only one-half the number that are required in single shear are necessary.

If two pieces of steel are placed close to-

gether as in Fig. 15, and pulled in opposite directions as indicated by the arrows, the rivet might shear off at x. If the rivet were too strong to shear off, it would pull out one or both of the plates, and therefore the joint would look either like Fig. 16 (a) or like 16 (b) after it had failed. When such a failure happens, the

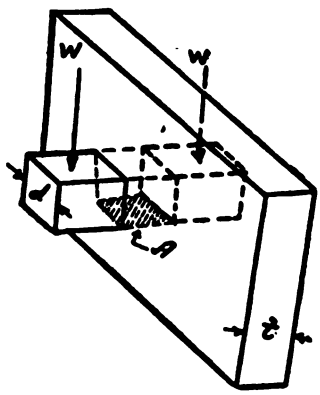


Fig. 17. Calculating Allowable Bearing Pressure on Joint.

joint is said to have failed by **bearing.** This means that the pressure of the rivet on the plate was too great, and it pushed its way through the plate, tearing an oblong hole as it went. If the rivet were square in section, it would not take any greater force to pull it through the plate than it does when the rivet is circular in section. The allowable amount of pressure (v) caused by a square or a round pin on a plate of a given thickness, is equal (see Fig. 17) to the area (A) in square inches multiplied by the allowable unit bearing stress (S_b). That is: $v=A\times S_b$, or, more simply, $v=AS_b$.

To get at this result in a still simpler way, the bearing value of a rivet (v_b) can be found by taking the allowable bearing stress (S_b), multiplying by the thickness (t) of the plate; and then multiplying the result by (d), the diameter of the rivet or length of side of the bar. This is true since the area A is equal to the product of the thickness of the plate times the diameter of the rivet or side of bar. In other words, since $A=t\times d$, we have the following simple rule for finding the bearing value of a rivet: $v_b=t\times d\times S_b$, in which v_b=bearing value of the rivet, in pounds; t= thickness of the plate, in inches; d=diameter of the rivet, in inches; and S_b=unit allowable bearing stress.

In (C 195-196) are given the bearing values of different rivets in plates of different thicknesses, and these are given for unit bearing stresses of 12,000, 15,000, 20,000, and 25,000 pounds respectively. If the bearing value of a $\frac{5}{8}$-inch rivet in a $\frac{1}{2}$-inch plate is desired when the unit-stress is 20,000 pounds, $\frac{5}{8}$ is noted in the first column of the third table; then that same line of figures is followed over until the column headed $\frac{1}{2}$ is reached. Here 6,250 is found, and this is the desired value. As a check here, $t=\frac{1}{2}$, $d=\frac{5}{8}$, and $S_b=20{,}000$. Therefore, the bearing value

$$v_b=\tfrac{1}{2}\times\tfrac{5}{8}\times 20{,}000=\frac{100{,}000}{16}=6{,}250.$$

Some blanks are left in the tables. The rea-

son for this is that it is not practicable to use rivets of the smaller diameters in the thicker plates. However, if it were required to obtain a bearing value not given, it is easily calculated from the rule already given. For example, let it be required to find the bearing value of a $\frac{5}{8}$-inch rivet in a $\frac{3}{4}$-inch plate when the allowable unit-stress is 25,000 pounds. Here $t=\frac{3}{4}$; $d=\frac{5}{8}$, and $S_b=25{,}000$; therefore the bearing value $v_b=\frac{3}{4}\times\frac{5}{8}\times 25{,}000=11{,}719$, say 11,720 pounds. It will be noted that this result is the same as if we had a $\frac{3}{4}$-inch rivet in a $\frac{5}{8}$-inch plate, as shown directly by the table.

In case the thickness of the plate is given in decimals of an inch, the same rule must be followed for determining the value of the rivets, since no tables are given other than those in (C 195-196). For instance, suppose that it is required to rivet something with $\frac{7}{8}$-inch rivets to the web of a 12-inch 31.50-lb. I-beam. By referring to (C 97), and the 5th column, the thickness is found to be 0.350 inch. If the allowable unit bearing stress is 20,000 pounds, the bearing value of one rivet is $v_b=0.350\times\frac{7}{8}\times 20{,}000=6{,}125$ pounds.

If the joint is like those shown in Figs. 15, 16, and 17, the rivet is said to be in **single bearing** in plate t_1 or t. It is also said to be in single bearing in plate t_2. Evidently, the thicker the plate, the larger the value of one rivet, and the fewer rivets are required to sustain a given total stress. Therefore it is only necessary to ex-

amine a joint for the number of rivets required in bearing in the thinnest plate, and this number will evidently be amply sufficient for the thicker plate.

It often happens that a joint is as in Fig. 13 (b), the two side plates being of the thickness t_1, and the intermediate one being of a thickness t_2. In many cases t_1 and t_2 are equal. In any case the rivet bears in each plate t_1 and in plate t_2 also. In such a case the rivet is said to be in single bearing in t_2, and in double bearing in t_1.

Just as in the case of double shear, the value of a rivet in double bearing is double that of its value in single bearing in the same plate. Also, it is determined in a similar manner. Thus the value of a $\frac{7}{8}$-inch rivet in double bearing in a $\frac{3}{8}$-inch plate (the unit bearing stress being taken as 25,000 pounds), is $2 \times 8{,}210 = 16{,}420$ pounds. See 4th table (C 195).

In making riveted joints, the hole is made slightly larger than the rivet. The rivet is then heated to a welding heat, and driven by a powerful machine which holds the heads tightly against the plate until the rivet cools. As the rivet is at a welding temperature when it has the other head formed, it soon cools to the temperature of the surrounding air, and, in doing so, contracts. If it cooled to 75 degrees the contraction would be so great as to cause a tensile stress in the rivet of over 30,000 pounds per square inch; and in the case of a $\frac{7}{8}$-inch rivet, whose section is 0.613 square inch, the total stress

would amount to over $0.613 \times 30{,}000 = 18{,}390$ pounds. This pulls the two heads toward each other, and, in doing so, clamps together very tightly the plates which are riveted together. The plates, according to tests made, are indeed clamped together so tightly that it takes a considerable force to start them slipping over each other, even when the rivets are not large enough to fill the holes.

The design of riveted joints will be taken up later.

TESTING OF STRUCTURAL STEEL

23. **Chemical Tests.** These are made in order to determine the percentage of the metalloids present in the steel—the chief of which are carbon, silicon, manganese, sulphur, and phosphorus. In making a melt of steel in the Bessemer or open-hearth furnaces, chemical tests are first made of the various parts of the charge and of the recarburizing material. A careful computation is then made as to how much of each of these is required to make steel of a given carbon and other metalloid content. During the progress of the open-hearth heat, small test bars are made at frequent intervals from the metal, and quick determinations of the carbon content are made. By this means it is ascertained when the carbon has burned down to the desired percentage; then the "recarburizer" is added and the material poured.

The making of chemical tests is of great im-

portance; but the **heat treatment** of steel—that is, the working of it when it is cooling—has such an effect on this material that steel of various physical properties can be made from a bath of the same chemical composition.

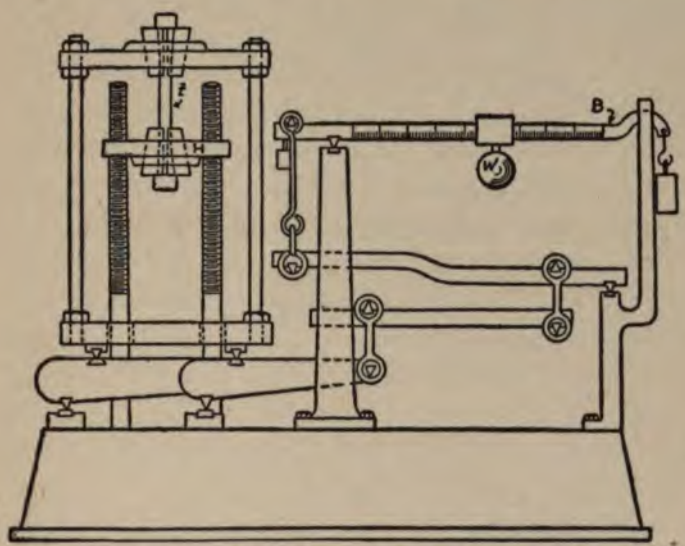


Fig. 18. Diagrammatic Representation of Testing Machine.

However, as most structural steel receives the same heat treatment, chemical analysis is a good guide to physical properties. Chemical tests are certainly invaluable in indicating the presence of sulphur and phosphorus.

24. **Physical Tests.** After the steel is manufactured, various physical tests are often insisted upon by the purchaser, and are conducted in order to make sure that the material is of the required strength and ductility. For it

is not always true that the stronger the steel the less ductile it is, although that is the case as a general rule.

Since the compression test is difficult to make, and since the tensile stress indicates closely what the result of the compressive test would be, the tensile test is the one usually employed. A piece of iron is put into a machine of which Fig. 18 is a diagrammatical representation. The vertical screws drive the head **H** in a vertical direction. The bar to be tested is put in at **t**; and the machine, which runs by hand or power, is started. The head **H** moves downward; and at last the test piece, which is held together tightly by the clamps, is pulled apart. The scale beam **B** is kept balanced by the operator moving the weight **w** backward or forward, and the amount of pull on the piece is registered at all times during the pull.

The highest value registered is called the **total strength** of the bar; and if this amount is divided by the area of the cross-section of the bar, there is obtained the **ultimate unit-stress**—often called simply the **ultimate strength.** As the area of the bar is measured in square inches, and the total strength is in pounds, it follows that the ultimate strength is in pounds per square inch. For example, if the bar was $\frac{5}{8}$ inch in diameter, and the highest value registered on the scale was 18,715 pounds, the ultimate strength would be $18{,}715 \div 0.3068 = 61{,}000$ pounds per square inch. The value 0.3068 is found in

column 5 (C 261), and is the area of a $\frac{5}{8}$-inch bar. If the bar were $1\frac{3}{8}$ inches in diameter, and the beam registered 86,273 pounds, the ultimate strength would be 86,273÷1.4849=58,100 pounds per square inch, the 1.4849 being the area in square inches of a $1\frac{3}{8}$-inch bar, which is found in column 5 (C 261).

The ultimate strength may be defined as the force required to pull apart a bar one square inch in cross-section.

At first, as the pull on the bar in the machine increases, the bar stretches at a uniform rate; that is, for every pound increase in weight, there is a corresponding amount of stretch, and this amount is constant for each pound increase up to a certain point. When a certain pull on the bar is reached, the amount of stretch per pound of tension changes quickly, and is considerably greater than that previously shown, being 1,200 to 1,300 times as great. Not only is the amount greater, but the rate of increase is variable, becoming greater and greater as the test progresses, until, when the bar breaks, the stretch for 1,000 pounds of pull of the machine is about 1,500 times what it was at first. The amount of stretch—or **elongation,** as it is called— is expressed in percentage of the length over which it is measured, and it is an indication of the ductility of the metal or its ability to bend considerably without cracking. The percentage of elongation for any certain class of steel is practically constant, being almost independent of

the size of the section, provided the measured length is the same.

The rate of increase in inches per amount of load is constant for all lengths, sizes, and classes of steel up to a certain point—namely, where the rate of increase in elongation changes. That is, the elongation, divided by the number of pounds which produce that elongation, gives practically the same result for all steels. This result is 30,000,000, and is called the **modulus of elasticity**—that is, the measure of elasticity. It signifies that the elongation, for each inch in length up to a certain point, is, when measured in inches, equal to one thirty-millionth of the load in pounds per square inch of cross-section of the bar. It is expressed in the following manner:

$$\Delta=\frac{Pl}{AE}$$

in which,

Δ=Total elongation in inches in a length of l inches;
l=Length over which the elongation is desired;
P=Load on the bar;
A=Area of the bar in square inches; and
E=30,000,000.

For example, if a bar $\frac{7}{8}$ inch in diameter is 18 feet long, and it is known to carry a stress or load of 18,030 pounds, the total stretch would be:

$$\Delta=\frac{18{,}030\times18\times12}{0.6013\times30{,}000{,}000}=0.216 \text{ inch.}$$

Here the 18 feet is reduced to inches by multi-

plying by 12, and the 0.6013 is the area of a $\frac{7}{8}$-inch round rod (C 261).

After the rate of elongation has changed from what it was at the start, the steel has begun to fail. This point occurs when the stress gets to be from $\frac{1}{2}$ to $\frac{3}{4}$ of the ultimate, and the

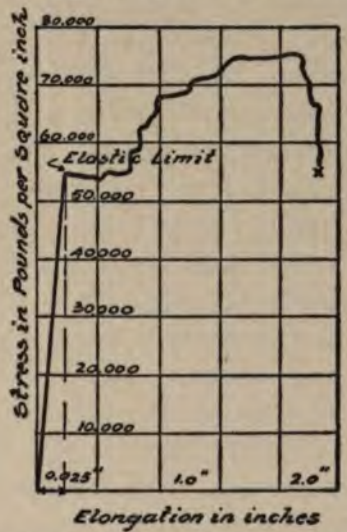


Fig. 19. Automatic Diagram of Tensile Test of Steel.

point is called the **elastic limit.** Mark this well, and note that **no allowable stresses should ever be above the elastic limit.** It is immaterial what the ultimate strength of a material may be; the **elastic limit** is what the engineer or the designer must know, and he must realize its importance.

If the elongations are measured by delicate instruments (called **extensometers**), and the readings of the scale beam are taken at the same time, the machine of course being stopped while

the readings of the extensometer and the scale beam are being taken; and if the elongations are plotted or laid out on a diagram from left to right along the horizontal, and the pounds per square inch are plotted along the perpendicular, lines drawn from these points will meet in a series of different points; and a line drawn through these points will give a curve, Fig. 19, which will show the elastic limit and the ultimate strength. Sometimes the machines have instruments attached that draw the curve while the test is going on. Fig. 19 is a curve of an actual test. The total elongation in 8 inches is seen to be 2.1 inches, and the percentage of elongation is therefore $2.1 \div 8 = 26.25$, which is better than is usually specified. The elastic limit is seen to be 54,500 pounds per square inch, and the ultimate strength is 75,000 pounds per square inch. These values are too high for structural material, although the test piece was a piece cut from a ⅜-inch rod. Evidently the rod got too cool during the last few passes in the rolls, and therefore it was cold-rolled to some extent. This would account for its rise in strength.

Fig. 19 shows that just after the elastic limit is passed the material stretches out very fast, and that the load for a little while is less than it was at the elastic limit. This is called the **yield point**, and in commercial testing is sometimes but incorrectly spoken of as the "elastic limit." The yield point may be determined

Courtesy of American Bridge Co.

SUSPENSION BRIDGE OVER NIAGARA RIVER AT LEWISTON, N. Y.

View taken from American side. Span length of stiffening truss, 800 ft.; span between towers, 1,040 ft.; span between anchorages, 1,200 ft. Weight, 1,840,000 lbs. Built in 1899.

without the use of extensometers, since the beam on the testing machine will drop down for a few seconds. A common operation is the determination of the yield point by the **drop of the beam.**

When the ultimate strength is reached, the

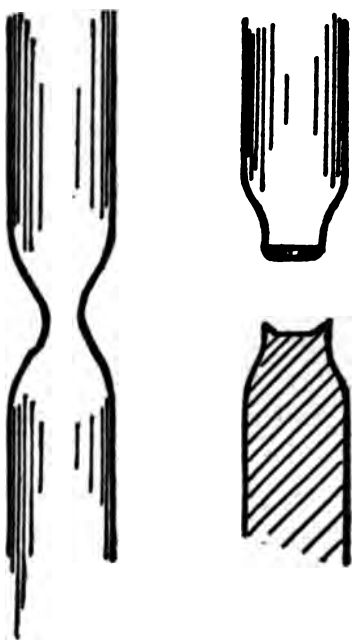


Fig. 20. Fig. 21.
Showing Necking-Down Action.

bar begins to **neck down** rapidly at some point, and quickly breaks, sustaining a load just before fracture about equal to the elastic limit. A necking-down point is shown in Fig. 20; and the two pieces just after fracture are shown in Fig. 21. The lower half is shown in section in order to show the cup-shaped fracture more closely. This fracture is characteristic of medium steel.

Soft steel does not show it; while hard steel shows little if any necking down, and breaks straight across.

This reducing is called the **reduction of area,** and is an index of the malleability of the material. The amount of reduction of area is expressed as a percentage of the area of the original cross-sectional area. Thus, if the bar was $\frac{3}{4}$ inch $= 0.750$ inch in diameter before testing, and 0.432 inch in diameter at the fractured part, the following is true:

Original area of 0.750-inch bar (C 261)=0.4418 square inch;

Area of fractured end of 0.432-inch bar (C 289)=0.1466 square inch;

Reduction of area=0.4418—0.1466=0.2952 square inch;

Reduction of area in per cent=0.2952÷0.4418=66.8, which is very good for structural steel.

Where the materials are subject to shock, as in case of railroad rails, the material, after it has been formed into the completed article, is subjected to an impact test. This consists in letting a certain weight drop from a certain height and strike the test piece, which is usually a beam, in the center. The amount of deflection is then measured, and the beam is struck another blow. The deflections under succeeding blows are measured, and the general distortions are observed. From this data can be deduced an idea of the ability of the article to withstand impact, which is another name for the striking of a blow or blows.

25. **Factor of Safety.** Now that the subject of ultimate strength and elastic limit are understood, the subject of the "factor of safety" should be taken up. Since the strength of a structure is no greater than that of its weakest part, and since, even with the best of care in the manufacture of steel, some that is not nearly so strong as the remainder will find its way into structures, and also since at times structures will be loaded far beyond those loads for which they were designed, and the loss of life and money caused by the collapse of a structure is likely to be very great, it is therefore customary not to take any chances of having a collapse, but to make the structure much stronger than it really would need to be if the quality of the steel were known to be the same throughout. The next question is, how many times too strong shall it be? Is it 2, 3, 4, or 5, or even 6? It is seldom 2, often 3, 4, and 5, and sometimes 6. In these figures we have what is called—but inadvisedly, as will soon be seen—the **factor of safety.**

The stress which is used in designing members is called the **allowable working stress,** or simply the **allowable stress,** and is obtained by dividing the ultimate stress by the factor of safety. The word **unit-stress** is understood in both cases as the stress in a certain area taken as a unit—a certain number of pounds per square inch.

If the stress which comes upon a member is

TABLE V

Factors of Safety and Allowable Stresses

Ultimate Strength of Medium Steel, 60,000 Pounds per Square Inch

Class of Work	Class of Stress	Factor of Safety	Allowable Working Stress
Buildings	Dead and Live Load	3	20,000 lbs. per sq. in.
Highway Bridges	Dead Load	2.4	25,000 " " " "
	Live Load	4.8	12,500 " " " "
Railroad Bridges	Dead Load	3	20,000 " " " "
	Live Load	6	10,000 " " " "

caused by a constant load—such as the dead weight of the structure, or a concentrated load which remains the same—then the factor of safety is small, because the conditions are quite well known. If the structure is subjected to moving loads which cause considerable jarring, then the factor of safety must be greater than in the former case. If the jarring is violent, then the factor of safety must be high. Table V gives factors of safety and allowable unit-stresses for various classes of structures.

It should be noted that the ultimate strength is divided by the factor of safety to get the working stress. This should not be. The elastic limit should be divided by the factor of safety, and our so-called "factors of safety" would then be about one-half of what they now are, since the elastic limit in structural steel is one-half or a little over one-half of the ultimate strength. Since, when the metal has been stressed beyond the elastic limit, the member has started to fail, and there is not much of the so-called "safety" in it, it will be clear that when a factor of safety of 4 is used, it does not

TABLE VI

Standard Physical Requirements of Structural Steel

(Theodore Cooper)

Class	Process	Ultimate Strength (Lbs. per sq. in.)	Lower Limit at Elastic Limit (Lbs. per sq. in.)	Reduction of Area	Lowest Value Per Cent of Elongation
Medium...	Open-Hearth	60,000–68,000	30 000–34,000	.....	22% in 8 in.
Soft.....	" "	54,000–62,000	27,000–31,000	.....	25% in 8 in.
Rivet Steel	" "	50,000–58,000	25,000–29,000	.....	26% in 8 in.

TABLE VII

Tests of Structural Steel

(Johnson, p. 43; Campbell, pp. 249 and 229)

Class	Process	Ultimate Strength (Lbs. Per sq. in.)	Elastic Limit (Lbs. per sq. in.)	Reduction of Area	Per Cent of Elongation
					8 Inches
Medium...	Open-Hearth	65,911–66,679	46,679–45,261	54.98–54.05	26.48–28.22
Soft.....	" "	58,130–62 840	41,400–43,600	60.1–63.1	29.25–30.00
Rivet Steel	" "	53,510–57,670	35,900–37,640	62.87–66.46	30.50–32.75

signify, as it was once thought to do, that the structure is four times as safe as it would be required to be if it were certain that the steel was the same throughout, but it really means that it is only twice as safe.

This point has been called to the attention of engineers time and time again; but while they recognize the meaningless character of the present so-called "factor," it has been found hard to change it, on account of the permanency which custom has imparted to it.

26. **Physical Requirements.** The physical requirements for soft mediums and rivet steel according to the common practice of to-day, are given in Table VI; and tests of structural steels are given in Table VII. It will be noticed that

the reduction of area is not a requirement in Table VI. This is due to the fact that the percentage of elongation indicates the same thing.

27. **The Test Piece.** Since probably from 75 to 85 per cent of the total elongation is within the limits of the necking-down part of the bar, and since this portion of the elongation is constant for bars of the same diameter or rectangular cross-section, it is of prime importance that the length of the test piece over which the elon-

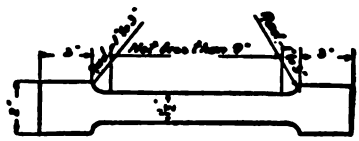


Fig. 22. Form and Dimensions of Steel Plate for Testing.

gation is measured be stated, and, moreover, that the length should be standard in order that different tests may be directly comparable as regards percentage of elongation shown.

Experience has shown that a length of eight inches is convenient, and this length is now most generally used. The relation of the length to the diameter of the test piece, or, if it be of square or rectangular shape, to the dimensions of the cross-section, affects the reduction of area. The ultimate strength and the elastic limit are also affected by both the area and the dimensions of the cross-sections. Hence it is important that test pieces should be standard. As yet, however, no one shape has been universally adopted, although commissions and technical

societies of France, Germany, and America have each proposed as standards test pieces whose dimensions differ very little among themselves.

Where nothing is said to the contrary, structural steel is tested with a measured length of eight inches, and the shape of the test piece is nearly always that recommended by the American Society for Testing Materials. This is the result of the best thought and experience of the foremost steel men of this country, and, as such, should be used in all cases.

Fig. 23. Microscopic Appearance of Absolutely Pure Iron.
Crystals closely packed together.

Fig. 22 gives the form recommended for plates. In case of round specimens the measured length is the same; but the diameter, if possible, is taken as equal to one inch, or, in case any other diameter is used, certain increases or decreases are allowed in the variation of ultimate strength, elastic limit, and percentage of elongation and reduction of area.

TABLE VIII

Physical Properties of Cast Iron, Wrought Iron, and Steel

Material	Ultimate Strength (Pounds)		Elastic Limit (Pounds)		Reduction of Area (Per Cent)	Elongation in 8 Inches (Per Cent)
	Tension	Compression	Tension	Compression		
Cast Iron	15,000– 18,000	80,000–200,000	15,000*	15,000	Slight	Less than 1
Wrought Iron	45,000– 55,000	45,000– 55,000*	25,000–35,000*	25,000	8 to 40	12.0 to 30.0
Steel	50,000–150,000†	50,000–150,000	110,000	35,000–110,000	62 to 25	20 to 11

* Average. † Steel has been recorded as high as 390,000 lbs. per square inch.

For further information on this subject the reader is referred to the Proceedings of the American Society for Testing Materials; to "Materials of Construction" (J. B. Johnson), and to "Manufacture and Properties of Structural Steel" (H. H. Campbell).

28. **Microscopic Tests.** If the surface of a piece of steel is polished and then treated with certain chemicals, certain parts of its surface will remain unaltered. Pure iron will be unattacked, while compounds of iron and the metalloids will be darkened.

The result as a whole cannot be observed with the naked eye; but, if put under a microscope and magnified about 500 times, the effect is remarkable. Fig. 23 shows what absolutely pure iron would look like. It shows the crystals closely together. When a small amount of carbon is in the iron, it does not, as

was formerly supposed, distribute itself through the total mass uniformly, but it combines in large percentages—say 0.9 per cent—with a small portion of the iron; and the resulting substance, which shows dark under the microscope, is found here and there between the masses of pure iron; see Plate 6 (a). As the percentage of carbon increases, larger proportions of the pure iron combine with it, and the dark places increase until the percentage of carbon rises to about 0.9 per cent, when the iron appears darkened all over; see (b), (c), and (d), Plate 6. In case other metalloids besides carbon are present, the iron may take up a larger percentage of carbon than 0.9 per cent; and then, while the carbon content of the steel may be larger, it will have more of free, pure iron in it than the lower-carbon steels. Plate 6 (e) (Institute of Civil Engineers, Vol. 123, 1896) shows such a case.

These views in Plate 6 are homogeneous; that is, the arrangement of the "carbon iron cement," if it may be so called, occurs with fair regularity. If, however, the steel is poor, the microscope will show irregular-sized crystals or long, dark places. At (f), Plate 6 (from "Engineering") shows some flaws.

29. **Physical Properties of Metals.** There are many different grades of cast iron, wrought iron, and steel; and their ultimate strengths, elastic limits, and percentages of elongation vary considerably, even in the same class of

material. Table VIII shows the variation in physical properties.

CLASSES OF STRUCTURAL MEMBERS

Beams and Structural Girders

30. The simplest member is the **beam** or **girder**. The terms "beam" and "girder" practically signify the same, and are so used by

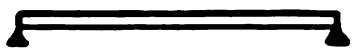


Fig. 24. Simple Beam.

many persons. It is better, however, to make a distinction, and to designate a member which is not **built up**, and which is the same size throughout, by the term **beam**. Those members which are composed of one or more beams, or of various combinations of plates and shapes, and which may or may not be of same size throughout, should be called **girders**.

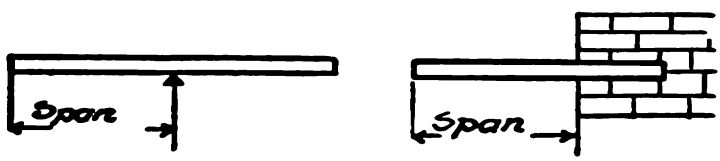


Fig. 25. Cantilever Beam.

The same formulas for stress apply to both beams and girders, but the method of design is different.

A **beam** or **girder** may be defined as a member which is supported on one or more supports, and which is stressed by being bent.

31. **Classes of Beams.** Beams or girders

are divided into several classes, according to the manner in which their ends rest or are fixed on the supports. These classes are:

1. **Simple beams,** or beams which have the ends resting freely on two supports (Fig. 24).

2. **Cantilever beams,** or beams which rest on one support only. Ofttimes cantilever beams have the second half built in a wall, one half only being used (Fig. 25).

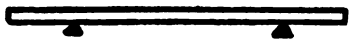


Fig. 26. Overhanging Beam.

Fig. 27. Continuous Beam.

3. **Overhanging beams,** which are a combination of classes 1 and 2 (Fig. 26).

4. **Continuous beams,** or beams which rest on more than two supports (Fig. 27).

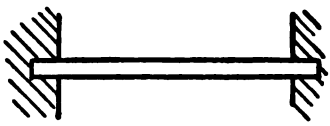


Fig. 28. Restrained Beam.

5. **Restrained beams,** which rest on two supports, but have their ends fixed so that they are immovable (Fig. 28).

6. **Hybrid beams,** which are beams combining the characteristics of two or more of the above classes of beams.

Simple beams are used to a far greater extent than all other classes put together. In their case, it is comparatively easy to analyze and calculate stresses; and it requires no heavy construction to hold the ends of such beams, since

they are perfectly free to move on their supports.

Overhanging beams are seldom used. They are, as will readily be seen, a combination of simple beams with cantilevers at the ends. The method of determining the stresses in them is more complicated than in the case of simple beams; but the design of overhanging beams—and, in fact, of all the other types of beams—is as simple as the design of the simple beam itself, once the stresses are obtained.

Continuous beams are seldom used. The computation of the stresses in them is quite complex. Also, the supports must always remain at the same elevation as when first built; for, if these should happen to rise or sink, extra stresses would occur in the beam, and, should the difference in elevation be considerable, the stresses might become so great as to cause failure or rupture of the beam.

On account of the fact that it is practically impossible to make foundations which will not settle to at least some small extent, beams of this class should never be used where they will rest on any artificial foundation. Theoretically, continuous beams are more economical than simple beams; but the fact that the foundations settle causes such uncertainty, and also such excessive stresses, that their use has been almost discontinued, except in cases where foundations are on the natural solid rock.

Restrained Beams. This class is open to the

same objection as continuous beams, for, if the supports should settle, the stresses would be increased above those computed. Theoretically, restrained beams require less material to support a given load than do simple beams, but the uncertainty of the stresses resulting from the un-

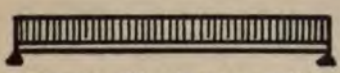


Fig. 29. Simple Beam with Uniform Load.

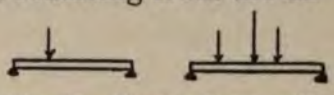


Fig. 30. Simple Beams with Concentrated Loads.

known amount of settling of supports, which is bound to occur unless the foundation is on solid rock, has either limited their use to such cases or they are made the same size as would be required for simple beams of the same span.

It is customary when a beam is partially fixed or restrained at the ends, to consider it as a simple beam, and design it as such. Then it is cer-

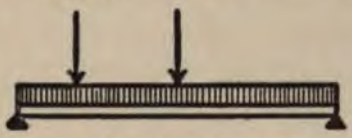


Fig. 31. Simple Beam with Both Uniform and Concentrated Loads.

tain that it will be safe, since, for a given span—that is, the distance between any two supports—and a given load, the simple beam requires more material than any other class.

32. **Loading of Beams.** Beams may be loaded by a **uniform load**—that is, a load of so many pounds for each foot of length; or they may be loaded by **concentrated loads**—that is, by

loads that come in certain places, somewhat as if a person were to set loads of given weights at certain points. A beam may have one or more concentrated loads upon it. It may also have a uniform load and concentrated loads on it at the same time. Figs. 29, 30, and 31 show these different methods of loading, and how they are in-

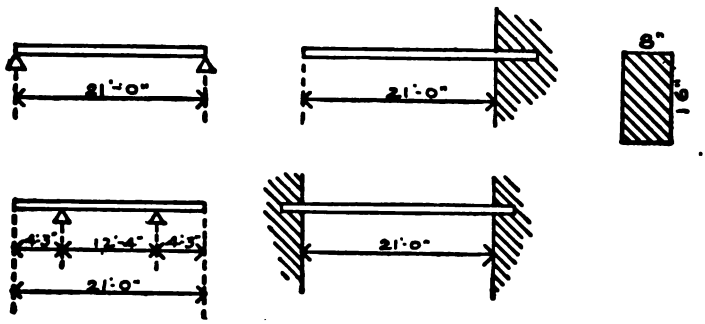


Fig. 32. Comparison of Different Classes of Beams with Equal Cross-Sections.

dicated in drawings. In case a beam has both concentrated and uniform loads on it at the same time, the stresses due to the two classes of loads are worked out separately; and then these stresses are added together for the design.

A wheel on a beam is considered as a concentrated load, since it touches the beam at one point only. Wheel loads are usually designated as **wheel loads**, to distinguish them from concentrated loads, since they can be moved about to any and every point on the beam, while concentrated loads cannot. Concentrated loads, in fact, are often spoken of as **fixed loads** for the very reason that they cannot be moved.

33. **Strength of Beams.** The problem of comparison of beams must be made with those having an equal number of supports and either carrying the same load or being of the same size. In the first case, the **sizes** are compared; while in the second case, the **carrying capacities** are compared.

For example, let four wooden beams be taken which are as in Fig. 32, and let them all be 8 inches wide and 16 deep. It is required to show how much each can safely carry. If they are of oak, the wood can be stressed up to 2,000 pounds per square inch, and still be safe, provided the load is a fixed one. The method of designing a beam is given in Article 38. Let the load be uniform in each case. Table IX gives the results.

TABLE IX

Comparison of Strength of Beams of 8 Inches by 16 Inches in Section

(See Figure 32)

Class of Beam	Span (Feet)	Total Load (Pounds)	Load per Linear Foot (Pounds)	Relative Strength
Simple Beam	21	21,680	1,033	1
Cantilever	21	5,420	258	¼
Overhanging	12.4 2-4.3	62,160	2,960	2¾
Restrained	21	32,500	1,550	1½

It will be seen that the cantilever beam is the weakest, and that the overhanging beam is the strongest, while the restrained beam is in between. Other considerations, however, make the simple beam the most economical. The ob-

jection to the cantilever is that it is far too weak for a given section. The overhanging beam would tend to fly up at one end when the load first came on it; and after the load was in the middle, the ends would tend to fly up. This would require additional expenditure in the first case, to hold the beam down to the supports; and in the second case, the moving up and down of the ends would keep this class of beam from being used in any work except when the load was on it all the time. The objection to the restrained beam is that if either of the supports settled while the load was on it, the beam would be stressed above what was allowed, and perhaps made to fail.

In case it is impossible to secure a second support, the cantilever must be used; otherwise it is inadvisable.

From the foregoing, the following rule may be deduced: **Use a simple beam in all cases where possible. If the beam is of the restrained variety, treat it as a simple beam,** and then the settlement of the supports will not stress it enough to make it dangerous.

The **stiffness** of beams is sometimes an important factor.

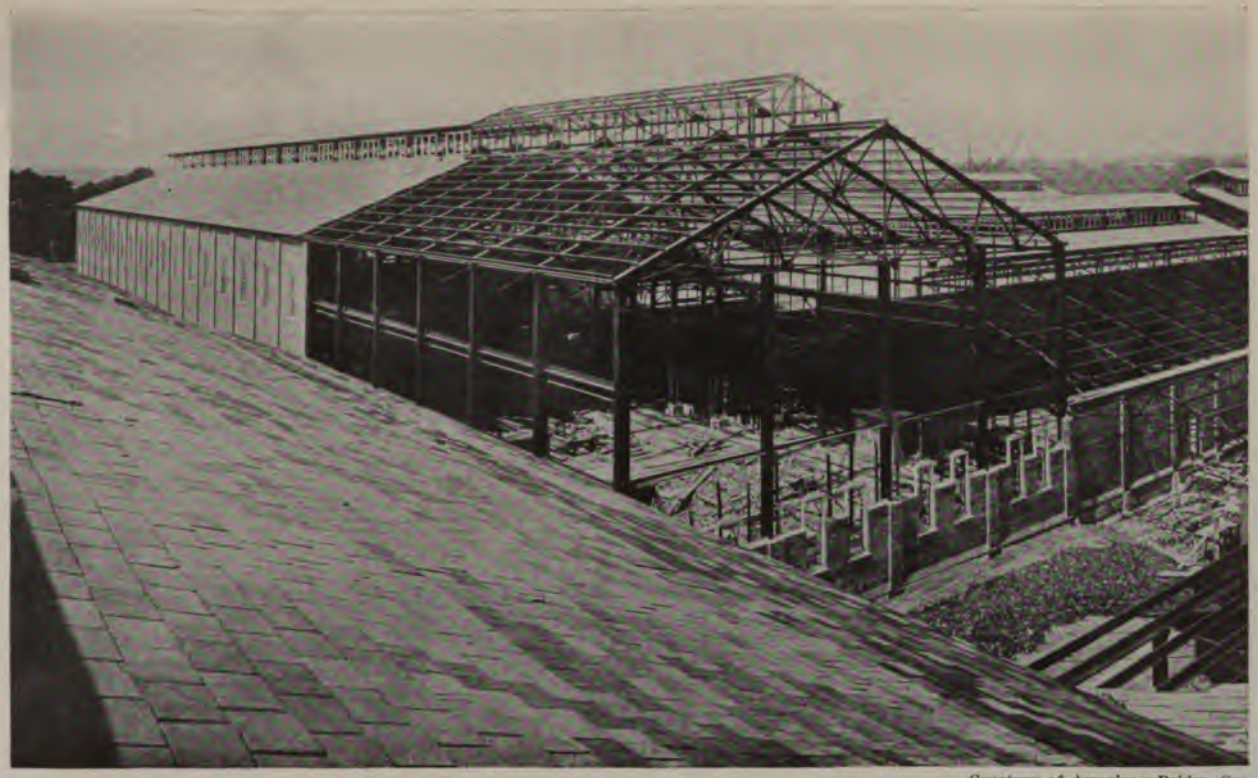
The strength of a beam of given span depends upon its **depth** to a greater degree than upon its **width.** It is a well-known fact that a board will carry more when stood up on edge than when it is lying flat. The depth being increased will rapidly increase the strength of the

Courtesy of American Bridge Co.

PENNSYLVANIA RAILROAD BRIDGE OVER SCHUYLKILL RIVER, PHILADELPHIA, PA.

One double-track deck pin-connected span, 236 ft. Weight, 1,140,000 lbs.

PLATE 18—STEEL CONSTRUCTION.



Courtesy of American Bridge Co.

BUILDINGS FOR THE PITTSBURG PLATE GLASS COMPANY, CRYSTAL CITY, MISSOURI.

Grinder and Polisher building, 408 ft. by 532 ft.; Laying Yard building, 100 ft. by 578 ft.; Lehr building, 68 ft. by 475 ft., and 114 ft. by 171 ft.; Stripper building, 90 ft. by 408 ft.; Warehouse, 153 ft. by 304 ft., and 90 ft. by 173 ft. Total weight of steel work, 6,280,000 lbs.

PLATE 19—STEEL CONSTRUCTION.

beam, the width remaining the same. The following is true:

Increasing depth	2	times	increases	strength	4	times.
"	"	3	"	"	"	9 times.
"	"	4	"	"	"	16 times.
"	"	5	"	"	"	25 times.

It is not so with the increase in width, the depth remaining the same. The strength is doubled if you double the width; it is made three times as strong if you make the width three times as great; and so on.

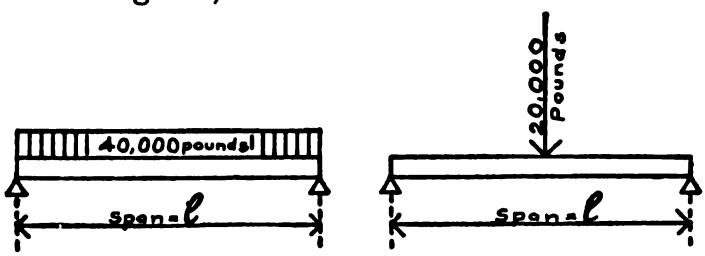


Fig. 33. Relative Capacity of Same Span when Load is Uniform and Concentrated.

If the size of beam remains the same, then the longer the span, the less load it will hold.

Making the span	2	times	as long	makes	the	strength	1/4	of	at first
" " "	3	"	"	"	"	"	1/9	"	" "
" " "	4	"	"	"	"	"	1/16	"	" "

And so on.

Any beam will carry just one-half as much load when the load is a concentrated load at its center, as it will if the load is distributed over the entire beam—that is, a uniform load. Fig. 33 illustrates this.

34. **Stresses in Beams.** When a beam is

subjected to a load, it bends; and in order that it may bend, the top part must shorten and the bottom part lengthen. Of course this shortening and lengthening is very small, and cannot be detected by the naked eye; but this distortion is what **stresses** the beam. If sections **a-a** and **b-b** were marked on the beam with a very sharp instrument (say a knife edge), and these sections were the same distance apart throughout their length (say 10 inches, see Fig. 34), and then, if

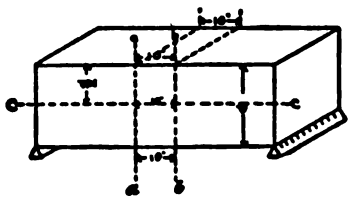


Fig. 34. Measuring Distortion in a Beam.

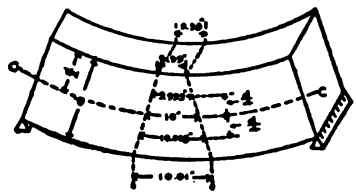


Fig. 35. Showing Distortion Due to Bending.

the beam were bent by loading it until computations showed that the most stressed parts of the beam were stressed to 2,000 pounds per square inch, and the distances between the two sections were then measured at the top and bottom, it would be found that the top would be almost one-hundredth of an inch shorter, and the bottom one-hundredth of an inch longer. Fig. 35 shows this, the bending being greatly exaggerated. If a line **c-c** had been drawn along the beam at its middle, and the distance between the two sections measured before and after bending, it would have been found out that the distance was the same on both sides.

Whenever a piece of material is shortened or lengthened, it is stressed. From Fig. 35 it will be seen that the material in the beam is stressed at the outer edges, and is not stressed at the center of the beam. It is also evident that at any point between the middle and the edges, the material is lengthened or shortened somewhat. This shows that the material is stressed at that point, but the stress is greater the nearer the point is to the outer edge. The distortion (that is, the shortening or the lengthening) at any point—and therefore the stress—is directly proportional to the distance of the point from the center. At one-half the distance out from the center, the amount would be one-half of what it was at the extreme edge (see Fig. 35). At all points on the line **c-c**, the stress is zero; that is, there is no stress. For this reason the axis indicated by this line is called the **neutral axis** of the beam.

35. **Reactions of Beams.** If a beam had any load placed upon it, the ends of the beam having been previously rested upon scales, the scales would register certain amounts. If the amounts each scale registered were added together, they would give a total equal to the weight of the beam and the load upon it. The amounts registered by each scale are called the **reactions**; that is, the reaction is indicated by the amount of pressure that the beam exerts upon the support. As the pressure of the beam under its loads is **downward**, the reactions must always

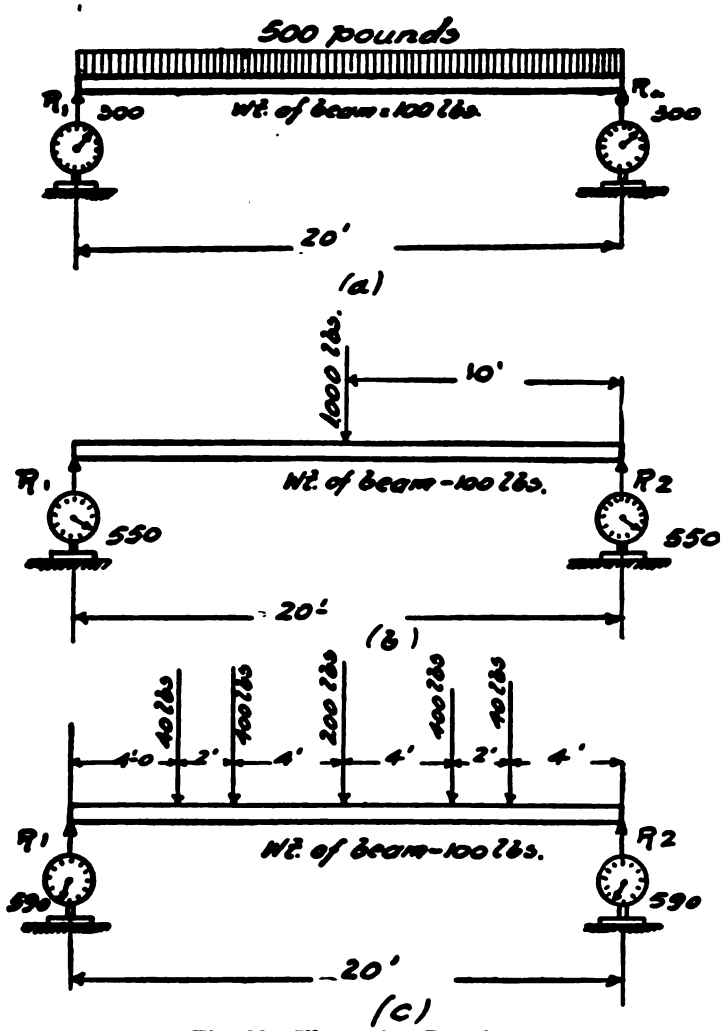


Fig. 36. Illustrating Reactions.

act **upwards**, in order to keep the beam in place. It is just the same—to take a simple example—as if one leaned over and pushed against a man; the man must push back if he expects to keep

the first fellow in place. It is a fundamental proposition or truth, that **the sum of reactions is always equal to the weight of beam and its loads.**

If the load is a uniform one, if the load is at the center, or if loads of equal weight are placed at equal distances on opposite sides of the center, then the reactions will be equal, and each one will be equal to one-half the weight of the beam and the loads on it. The reaction at the left

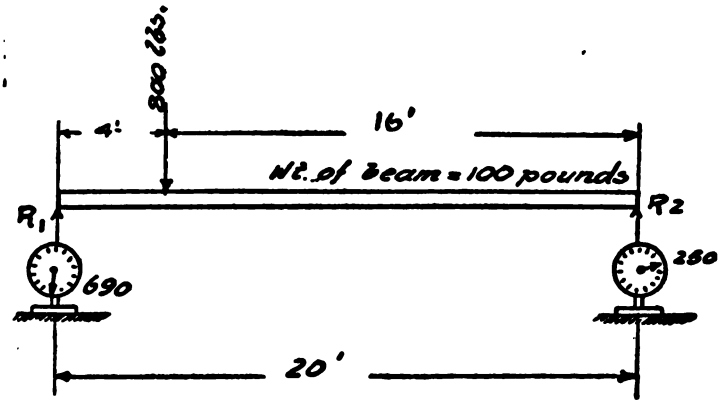


Fig. 37. Reactions in a Beam with Concentrated Load not at Center.

hand is usually called R_1, while that at the right is called R_2 (see Fig. 36). For example, if the beam weighed 100 pounds, and the loads were distributed as in Fig. 36 (a, b, and c), the reactions would be as follows:

In Fig. 36 (a), R_1=one-half of (500+100)=300 pounds; and R_2 would be the same.

In Fig. 36 (b), R_1=one-half of (1,000+100)=550 pounds; and R_2 would be the same.

In Fig. 36 (c), R_1=one-half of (40+400+200+400+40+100)=590 pounds; and R_2 would be the same.

If the loads are placed differently from the above, the reactions will not be the same. If one load is used, the reaction nearest to the load will be greatest; and the nearer the load is to the reaction, the greater will be the reaction. If the load is right at the support, the reaction will equal the load, together with one-half the weight of the beam. If the conditions were as in Fig. 37, the scales would register:

R_1=690 pounds
R_2=210 pounds
Sum=900 pounds

As the total weight is 800+100=900 pounds (the 100 pounds being the weight of the beam), this must be equal to the sum of the reactions, and is so, as shown.

The determination of the reactions can be made without the use of scales. The reactions can be computed. In cases like those in Fig 36, each reaction is, as before stated, equal to one-half of the load and the weight of the beam.

In cases like Fig. 37, or where there are more than one load irregularly placed or irregular in weight (Fig. 38, a and b), the determination is made as follows: Multiply each load by its distance from the end of the beam at which the reaction is not desired, and divide this by the span

of the beam. This will give the reaction due to the loads, and to it must be added one-half the weight of the beam. The result is the reaction.

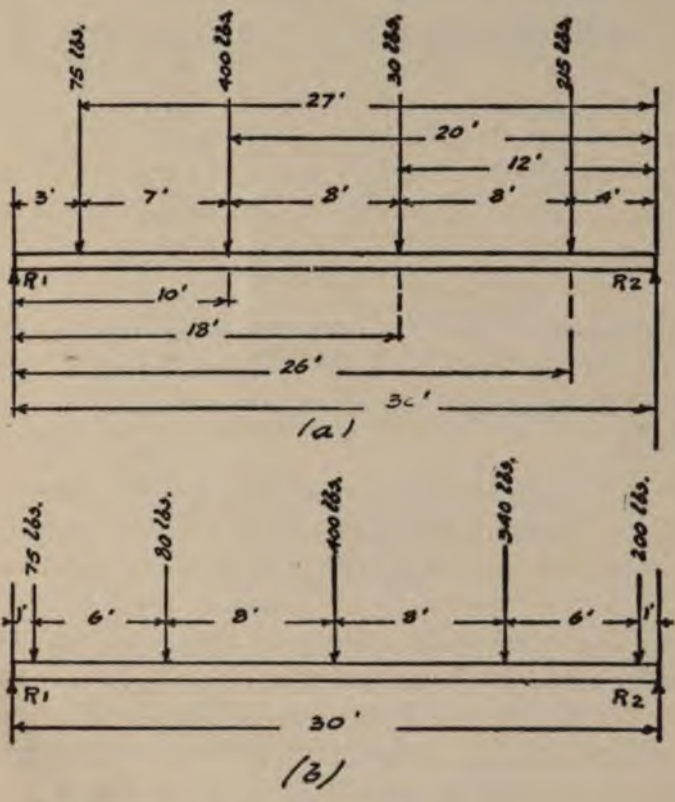


Fig. 38. Reactions in Beams Irregularly and Unsymmetrically Loaded.

For example, take Fig. 38 (a). The computations are as follows:

For R_1		For R_2	
215× 4=	860	75× 3=	225
30×12=	360	400×10=	4,000
400×20=	8,000	30×18=	540
75×27=	2,245	215×26=	5,590
30)11,245	(374 25/30	30)10,355	(345 5/30
90		90	
224		135	
210		120	
145		155	
120		150	
25		5	

Therefore R_1 due to loads=374 25/30; and R_2 due to loads=345 5/30. Now, one-half the weight of the beam is 100÷2=50 pounds; therefore,

R_1=394 25/30+50=424 25/30 pounds
R_2=345 5/30+50=395 5/30 "
Sum=820 pounds.

This 820 pounds is equal to the sum of the weights of all the loads and the beam.

When the sum of the computed reactions is not equal to the sum of all the weights which produce them, it is evident that an error has been made in computing them, and the work should be done again.

In Fig. 38 (b) the loads are placed symmetrically with respect to the center; but they are not equal in weight. The reaction R_1 is computed as follows:

200× 1= 200
340× 7= 2,380
400×15= 6,000
80×21= 1,840
75×29= 2,175

30)12,595(419 25/30 is R_1 due to loads
120
59
30
295
270
25

419 25/30=419.8 is R_1 due to loads
100÷2= 50.0 is R_1 due to weight of beam
Sum =469.8 is total R_1

R_2 is 725 5/30, and this should be worked out by the reader.

36. **Shear in Beams.** Suppose that in place of the window-sash a solid piece were put in, and that the sash weights were taken off and certain weights placed on the top. Also, in order to keep the sash from going clear down, supports were placed under the ends, Fig. 39, and pressed upward with pressures of 14.4 and 23.6 pounds. This piece is then in reality a beam; and the pieces of wood which support its ends are in reality pushing upward with a force equal to the reactions R_1 and R_2. The reactions due to the loads are R_1=14.4, and R_2=23.6. The weight of the beam is neglected in this case, as it is small. In all cases in this and the next article, the weight of the beam will be neglected,

since, in the design of beams, the beam is not known until after the design is made, and therefore the beam is designed to hold up the loads upon it, then the weight of this beam is computed, and then the beam is re-designed in order that it may carry both the weight of the loads and also its own weight. This method of procedure will appear clearer after it has been studied in Article 38.

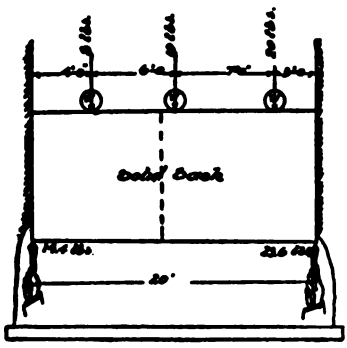


Fig. 39. Illustrating Shear.

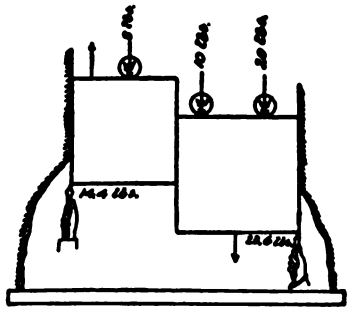


Fig. 40. Illustrating Shear.

Now, referring to Fig. 39, suppose that while the solid sash was still in the position, it were sawed clear through along the dotted line. Now what would happen? Why, since the left-hand end of one sawed part is being pushed upward by 14.4 pounds, and being pushed downward by the weight on top, which is only 8 pounds, the left-hand piece would move upward with a force of 14.4—8=6.4 pounds. The right piece has a force of 10 and 20, which is 30 pounds pushing it down, and only 23.6 pushing it up, and therefore it will go downward with a force of 30 less

23.6, which is 6.4 pounds, Fig. 40. Therefore one part will move upward, and the other part downward, as shown in Fig. 40, the parts sliding over each other along the section sawed. Before the material was sawed, the material at this place was strong enough to keep the pieces from sliding. In other words it was strong enough to stand the 6.4 pounds. On account of the fact that the portions slide over each other in a manner similar to being cut by a pair of shears, the action is called **shearing,** and the value 6.4 is called the **shear.** As long as the loads between the place considered and the end are less than the reaction at that end, that part will move or tend to move upward with a force equal to the difference between the reaction and the loads.

When the loads on top and between the place and the reaction are greater than the reaction, then that portion of the beam will move or tend to move downward.

In order to simplify the work of writing out certain portions of this work, the following signs will be used:

× means Multiplied by;
\+ " Added to (or positive);
\- " Subtracted from or taken from (or negative);
= " Is equal to, or comes to, or the amount left;
÷ " Divided by.

Thus 16+7=23 means that if 16 be added to 7, it will come to 23. Or again, 16-10=6 means

that if 10 be taken away from 16 there will be 6 left, or the amount left will be 6.

From the foregoing, **shear** may be defined as **tendency of one part of a beam to slide past the other part.**

The actual value of the shear is equal to the reaction R_1, less all the loads between the section (part under consideration) and the end at which R_1 acts.

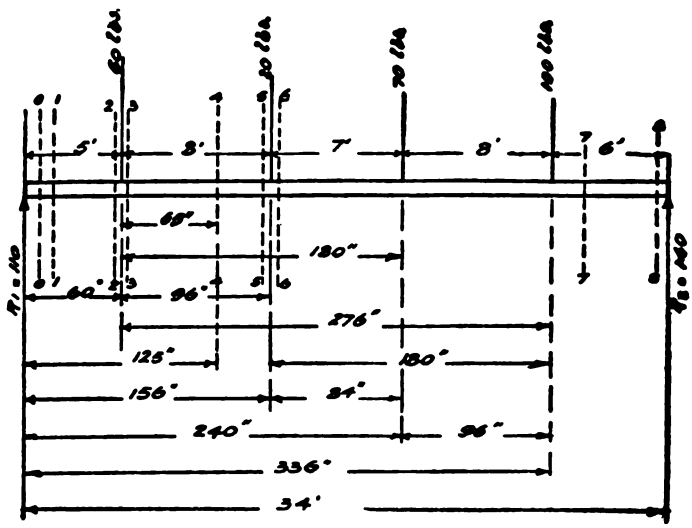


Fig. 41. A Beam with its Loads and Reactions.

In Fig. 41, a beam, together with its loads and reactions, is shown. At various points along its length are sections as 0-0, 1-1, 2-2, 3-3, etc. In cases such as 2-2 and 3-3, and 5-5 and 6-6, the sections are taken just as close to the loads as

possible, one just ahead of and one just behind it. The shears are computed as follows:

Shear at 0–0=110–0 (no load)	= +110	pounds
" " 2–2=110–0 (no load)	= +110	"
" " 3–3=110–60	= + 50	"
" " 4–4=110–60	= + 50	"
" " 5–5=110–60	= + 50	"
" " 6–6=110–(60+20)	= + 30	"
" " 7–7=110–(60+20+70+100)	= –140	"

By studying the above results, it will be seen that the shear at any place in the beam between the end and the first load is equal to the reaction at that end. Also, that the shear is the greatest at the ends of the beam, or between the end and the first load; and that the shear is the same at any point in the beam, provided that point lies between any two loads or between a reaction and a load.

Since beams are built of the same size all the way from one end to the other, it is only necessary to see if there is enough material in the cross-section to resist or stand the shear at the end, that is to stand a shear equal to the greatest reaction.

It will be seen that some of the values for shear have a + and some a — before them. This means that the left reaction is greater than the loads between it and the section, or that the left reaction is less than the loads between it and the section. It also signifies that in the first case there is a tendency for the left portion of

the beam to move up, and in the second case to move downward.

For a beam uniformly loaded the greatest shear is equal to one-half of the total load on the beam; that is, it is equal to either reaction, since these are equal.

Unless a beam is very short, say not longer than ten times the depth, it will be safe for shear if it is large enough to carry the loads.

37. **Bending Moment in Beams.** The **bending moment,** often simply called the **moment,** is that which causes the beam to bend or to break. The amount of bending moment depends not only upon the weight of the loads but also upon the position of them. It is well known that a man may stand out a short distance from the end of a small plank and be safe, whereas, if he were to go further towards its center, it would break and let him down. The rule is that the further a load gets toward the center of a beam, the larger the bending moment it will create.

The bending moment at any section is equal to the amount obtained by multiplying the left reaction by the distance from the section, and then taking from this value the amount obtained by adding together the results obtained by multiplying each load which is between the section and the left reaction, by its distance from the section. This is the rule for beams with concentrated loads. All distances should be in inches.

In the case of a uniform load the greatest mo-

ment occurs at the center of the beam, and is equal to the total load multiplied by the length of the beam measured in inches and then divided by 8.

In a series of concentrated loads, the greatest moment will occur under one of the loads. It is uncertain which load this is; and so the moments under each load, beginning at the left-hand load, should be computed until it is found.

Referring to Fig 41, the moments are as follows:

COMPUTATIONS				MOMENT
Under load	60 lbs:	110×60=		6,600
" "	20 " :	110×156—60×96=		11,500
" "	70 " :	110×240—(60×180+20×84)=		13,920
" "	100 " :	110×336—(60×276+20×180+70×96)=		10,080

This shows that the greatest moment is under the 70-pound load, and therefore the beam must be designed to withstand that moment.

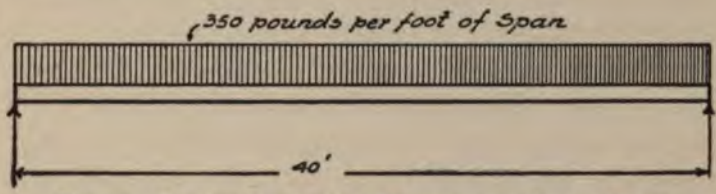


Fig. 42. Illustrating Bending Moment in a Simple Beam.

If a beam were as in Fig. 42, the total weight would be 350×40=14,000 pounds. The greatest bending moment would then be, according to the above instructions:

$$(14{,}000\times40\times12)\div8=840{,}000$$

The numerical operations are:

$$
\begin{array}{r}
14000 \\
\quad 40 \\
\hline
560000 \\
\quad 12 \\
\hline
1120000 \\
560000 \\
\hline
8)6720000(840000 \\
64 \\
\hline
32 \\
32 \\
\hline
0000
\end{array}
$$

This beam would then have to be designed to stand a bending moment of 840,000.

Since the bending moment is obtained by multiplying pounds by inches, it is spoken of as **pound-inches**; that is, the bending moments above would be spoken of as 13,920 pound-inches and 840,000 pound-inches, respectively.

It may be desirable to determine the bending moment at a point other than under a load. The method is the same as previously followed. For example, supposing it is desired to determine the bending moment (M) at section 4-4 of Fig. 41, it would be:

$$
\begin{aligned}
M &= 110 \times 125 - 60 \times 65 \\
&= 13{,}750 - 3{,}900 \\
&= 9{,}850 \text{ pound-inches.}
\end{aligned}
$$

By comparing this with the moments under load 60 and load 20, it will be seen that, while

Courtesy of American Bridge Co.

FLATIRON BUILDING, NEW YORK CITY, UNDER CONSTRUCTION.

At junction of Broadway, Fifth Avenue, and 23d Street. Twenty stories, including basement. Weight of steel used, 6,963,500 lbs.

PLATE 21—STEEL CONSTRUCTION.

it is greater than that under 60, it is smaller than that under 20. This brings to notice the fact that the moment, unlike the shear, is not the same between any two points when the beam is unsymmetrically loaded either according to the placing of the loads or their weights. This fact is also true in the case of a beam uniformly loaded.

The bending moment is usually designated by the letter M, and the shear by the letter V. If it is desired to designate at what section the moment or shear is taken, it is written thus:

	$M_{a\text{-}a}$	means	Moment at section a-a;
	$V_{b\text{-}b}$	"	Shear at section b-b;
or,	M_1	"	Moment at first point;
	V_2	"	Shear at second point, or in between first and second loads or points.

DESIGN OF BEAMS.

38. The design of beams is a simple matter, once the character, amount, and position of the loads that come upon them are determined.

The case of uniform loads will first be considered. The usual case is where a series of beams run side by side at a certain distance apart, and these carry the floor as in Fig. 43, which is a view looking down on top of the floor. They may be spaced equal distances apart, or they may not. In the figure, the span of all the beams is 20 feet, and the first three are spaced 5 feet apart, while the last is 8 feet from the third beam.

The loads that are to come on a beam are usually stated as so many pounds per square foot of floor surface, and the weight of the floor material is given in pounds per square foot. These values are different for different cases. The weights of floors can be ascertained from any architect. Some kinds of floors are shown in C 48-49. The weights of floor per square foot will vary from 10 pounds for simple two-thickness wood, to 50 pounds or more for reinforced concrete. The so-called **live load**, or load which may be put on the floor after it is built,

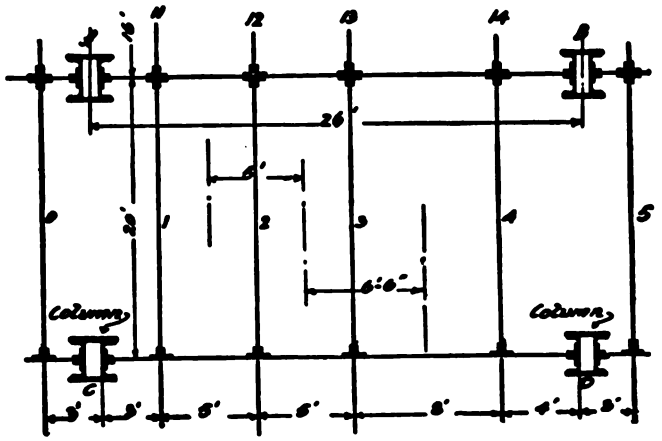


Fig. 43. Beam Framing.

will be given by the specifications. This varies from 100 pounds per square foot for ordinary office buildings, to 700 or 800 pounds or more for warehouse floors or floors on which heavy machinery is to be placed. All that is required, then, is to find out the number of square feet

there are on which if a load be placed the entire weight will come upon the beam in question, and then multiply this by the weight of the load on a single square foot.

To determine the number of square feet as above described, multiply the span of the beam by the sum of the distance to the beams on each side of it, and divide the result by 2.

For example, let it be required to determine the number of square feet on which, if loads were placed, the beam 2, Fig. 43, would be stressed.

$$20\times(5+5)\div 2=100 \text{ square feet.}$$

The computation is as follows:

$$\begin{array}{r} 5 \\ 5 \\ \hline 10 \\ 20 \\ 2\overline{)200} \\ \hline 100 \text{ square feet.} \end{array}$$

This may be stated another way as follows: Multiply the span of the beam by the distance from the center between beams on one side to the center between beams on the other side; or, if the beams are spaced equal distances apart, multiply the span by the distance between beams.

In case of beam 2 in Fig. 43, these last two rules give $5\times 20=100$ square feet the same as before.

For beam 3, Fig. 43, the spacing is unequal.

The first method gives $20\times(5+8)\div2=130$. The second method gives $20\times6.5=130$. The third method cannot be applied, since the spacing is not the same. Beam 1 has 110 square feet, and beam 4 has 150 square feet. The computation of this is left to the reader.

Supposing that the floor material is such that it weighs 15 pounds per square foot, and the specifications demand that the floor shall hold a load of 300 pounds per square foot, the total loads on the beams are now computed as follows:

Beam	Square Feet × Load per Square Foot	Total Load
1	110×315	34,650 pounds
2	100×315	31,500 "
3	130×315	40,950 "
4	150×315	47,250 "

First Method of Design. Once the **total load** is known, and the kind of beam that is to be used, the beam may be taken from the tables in the handbook if the specifications allow a unit-stress of 16,000 pounds. On pages 70 to 80 inclusive, Carnegie Handbook, are tables giving the number of tons a beam of certain kind, size, and span will carry. A ton is 2,000 pounds, and therefore the above values must be divided by 2,000 in order to reduce them to tons. These loads are:

Beam	Load
1	17.33 tons
2	15.75 "
3	20.48 "
4	23.63 "

If the beam is to be an I-beam, and beam 3 is to be designed, we must find the span 20 feet in the first column, and look across until we find a value of 20.48 or a little greater. On (C 70), under a 15-inch 60-lb., is found 21.65, and this shows that the beam we require is some place between this and the 15-inch 42-lb., which is the next column to the right and which gives 15.71 tons. By (C 97) it is seen that the I-beams between 42-lb. and 60-lb. are 45, 50, and 55-lb. sections. On pages 70-74, in the column to the right of the values of tons for any particular sized I-beam or channel, is the amount of a ton that can be added to the value given in the columns to the left in order that the tons for an I-beam or channel not given in these tables may be computed. The number of tons which the 45, 50, and 55-pound I-beams will hold are computed as follows, the span being 20 feet in each case (C 70):

Section	Difference from 42-lb.	Total Tonnage
45-lb.	45–42=3	15.71+ 3×0.20=16.31 tons
50 "	50–42=8	15.71+ 8×0.20=17.31 "
55 "	55–42=13	15.71+13×0.20=18.31 "

The 0.20 is obtained from the column to the right of the 15.71. Since all of these values are smaller than the 20.48 which the beam is to carry, they will not do. It is noted that these tables take into account the weight of the beam, and so this is to be deducted in order to get the

actual carrying capacity. The weight of the beam is $(60 \times 20) \div 2{,}000 = 0.6$; and the load, in addition to its own weight, which the 15-inch 60-lb. beam can carry is $(21.65 - 0.60) = 21.05$. The 15-inch 60-lb., with a carrying capacity of 21.65, including its own weight, comes the nearest to the required, and could be used.

This beam will be used if the depth of the beam is limited to 15 or 16 inches; but if it is not limited, it will be better to use an 18-inch 55-lb. beam. The table (C 70) shows opposite the 20-foot span a carrying capacity of 23.58 tons, and it should be noted that the weight is 5 pounds less than the 15-inch 60-lb. beam. Therefore the 18-inch beam is cheaper and should be used. This point is often overlooked by designers. After a beam is designed as closely as possible according to its carrying capacity, one should see if there is not a deeper and lighter, and therefore cheaper, beam which will carry as much or more weight.

If beam 4 is to be designed, it will be seen by (C 70) that an 18-inch 55-lb. carrying 23.58 tons (including its own weight) is too small. By (C 97) it is seen that there are also 18-inch beams weighing 60, 65, and 70 pounds. In the table (C 70), the column to the right of the 18-inch beam column shows that, for each pound above 55 pounds, the beam will carry 0.24 ton. The carrying capacities of the 18-inch beams are:

Beams	Difference in Weight	Total Carrying Capacity	Net Capacity
70-lb.	70–55=15	23.58+15×0.24=27.18 tons	26.48 tons
65 "	65–55=10	23.58+10×0.24=25.98 "	25.33 "
60 "	60–55= 5	23.58+ 5×0.24=24.78 "	24.15 "
55 "	= 0	23.58 + 0 =23.58 "	23.00 "

The **net capacity** is the capacity after the weight of the beam has been taken from the total capacity. Since beam 4 requires the net capacity to be 23.63 tons, it is evident that an 18-in. 60-lb. I-beam satisfies this condition, and will be used, since by looking at the next deeper beam, the 20-inch., it is found that there is no 20-inch beam which is lighter and which gives 23.63 tons net or more.

By consulting the tables for channels and other shapes (C 73-80), it will be seen that the loads above are too heavy to allow any other shape but an I-beam to be used. Of course, for beam 4, two channels might be used, and each would have to have a net capacity of 23.63÷2 =11.82 tons, or three with net capacities each of 23.63÷3=7.88 tons. By (C 73), opposite the 20-foot span of the first column, it is seen that two 15-in. 33-lb. channels are just too light, and that three 12-in. 20.5-lb. channels only give 3×5.70=17.10 tons. By (C 101) it is seen that there are five 15-in. channels whose weights are 35, 40, 45, 50, and 55 pounds. By the column (C 73), to the right of the 15-in. channel column, it is noted that 0.20 ton is added for every pound

weight of section. The 15-inch 35 and 40-pound channels have net carrying capacities as follows:

Channel	Difference in Weight	Total Capacity	Net Capacity
35	35 − 33 = 2	11.11 + 0.20 × 2 = 11.51 tons	11.16 tons
40	40 − 33 = 7	11.11 + 0.20 × 7 = 12.51 "	12.11 "

The required net carrying capacity for one beam is 11.82 tons. Comparing this with the above computations, it is seen that two 40-pound 15-inch channels are sufficiently strong. They are, however, very costly, when compared with the one 60-lb. 18-inch I-beam, since the two channels weigh 80 pounds, whereas the single I-beam weighs 60 pounds. In general it may be said that **when used as a simple beam, a single shape is more economical than two or more others.**

Second Method of Design. A second method may be employed to determine the size of the required beam. This is by the tables (C 97-106). In this case a little more latitude is allowed in the unit-stresses employed. Either 16,000 or 12,500 pounds per square inch may be used as desired, in case of beams and channels; while for Z-bars or T-shapes, 16,000 or 12,000 may be used. In this case the beam is designed by the use of the coefficients in the columns headed C and C′.

Rule—**For uniform loads, multiply the total load on the beam in pounds by the span in feet, and select the sized shape in the columns of the table which occur on the same line as the value**

under C or C′, as the case may be, which is equal to, or larger than, and as close as possible to, the value obtained by multiplying.

For example, let it be required to design beam 3, which has a total load of 40,950 pounds, the allowable unit-stress being 16,000 pounds per square inch. The span is 20 feet. For this case, the required coefficient $C = 40{,}950 \times 20 = 819{,}000$. Now, looking in the column "C," we find 866,100 as nearest to and larger than 819,000; and on the same line in columns 2 and 3, is found the 15-in. 60-lb. I-beam. This 866,100 is the total C for that beam, and from this must be subtracted the C due to the weight of the beam. The weight of the beam is $60 \times 20 = 1{,}200$ pounds, and its C is $1{,}200 \times 20 = 24{,}000$. This gives a net C of $866{,}100 - 24{,}000 = 842{,}100$. This beam could be used; but, by further inspection, it is found that C for an 18-in. 55-pound I-beam is 943,000. As this last beam is stronger as shown by the larger C, and lighter than the 15-in. beam, it should be used. This is the same result arrived at by the first method, and this method is much shorter.

Let it be required to design the same beam when the allowable unit-stress is 12,500 pounds per square inch. C′ is obtained in the same manner, and is equal to C—that is, $40{,}950 \times 20 = 819{,}000$. This time we must look in the 13th column, where C′ is given (C 98). The value next above 819,000 is 853,000, which is for an 18-in. 70-lb. I-beam, as noted by following the

same line over to the 2d and 3d columns. From this 853,000 must be subtracted the C′ of the beam. The weight of the beam is 60×20=1,200, and the C′ is 1,200×20=24,000. The net C′ for the 18-in. 70-pound beam is then 853,000—24,000 =829,000, which is larger than the required C′ 819,000, and therefore shows it to be strong enough. It will be noted that a 20-in. 65-pound I-beam gives a C′ of 974,700, which shows it to be stronger, and as it is also 5 pounds lighter than the 18-inch beam, it should be used unless the depth is limited to 18 inches for some reason.

Third Method of Design. A third method is one which can be used for all kind of beams, all kinds of loading, and all values of allowable unit- stresses. The great objection to the above methods is that they can be applied in case of uniform loads only, and that the allowable unit-stress is limited to one or two values. This third method may be divided into several steps, namely:

(a) Determine the greatest moment in the beam.
(b) Divide this by the allowable unit-stress.
(c) The result is the required section modulus, S.
(d) Look in the column headed S (C 98-120), and, finding a value for S equal to it or next greater in value, pick out the size of the shape in the columns on the left-hand side of the page.

Let it be required to design beam 3, Fig. 43, by this method, when the allowable stress is 16,000 pounds. According to the rule on page

175, the greatest moment is M=(40,950×20×12)÷8=1,228,500 pound inches. Then, by dividing this by 16,000, S is obtained. It is:

$$S=(1,228,500)\div 16,000=76.9.$$

By looking in the column headed "S" (C 98), the next highest value is 81.2, and this belongs to a 15-in. 60-pound beam. The S required for the beam must now be computed. The weight of the beam is 60×20=1,200 pounds; the moment is (1,200×20×12)÷8=36,000 pound-inches; and the S required is 36,000÷16,000=2.25. The net S for the beam in question is 81.20—2.25=78.95, which, being greater than the 76.9 required, shows that the beam is strong enough; but, as before, it will be noticed that the 18-in. 55-pound beam, with S = 88.4, is lighter and stronger, and therefore cheaper. It will be used.

In case the allowable unit-stress was 10,000 pounds per square inch, as is likely to be the case in railroad bridges, the S required is 1,228,-500÷10,000=122.85. This shows that a 20-in. 75-pound beam will probably do. This beam has a total S of 126.9, requires an S for its own weight of (75×20×20×12)÷(16,000×8)=2.81, thus giving a net S of 126.90—2.81=124.09, which is sufficient, as it is greater than the required S of 122.85.

Many engineers, in their specifications, allow that, in designing, the designer may neglect the weight of the beam if it is about one-tenth of the weight it carries. Under this condition, the

weight of the beam in all the above cases would have been neglected.

As has been mentioned before, the beam is usually safe to resist shear when it will carry the load in bending. Let beam 4 of Fig. 43 be investigated to see if it is safe to resist shear. The allowable unit-stress in shear will be taken as 10,000 pounds in this case. In any special case the specifications will give the stress allowed. The total end reaction of beam 3 is 40,950÷2=20,475 pounds (see page 163). This is the greatest shear (see page 174). The number of square inches required to resist the shear is:

$$20{,}475 \div 10{,}000 = 2.0475 \text{ sq. in.}$$

On page 187 the beam has been designed and found to be a 15-in. 55-pound I-beam. In column 4 (C 97), the area of the beam is found to be 16.18 square inches, and this is amply safe in this respect. This great difference between the required and that given is not unusual in the design of beams, but is a usual fact. Beams large enough to carry the load so as to prevent breaking by bending, usually show a great excess when examined for shear. See page 212 for further information on this point.

In some cases the moment is so small that either an I-beam, a channel, or a Z-bar may be used. In such case, the three are designed, and the lightest (and therefore the cheapest) is taken.

For example, suppose the moment in a cer-

tain beam was 125,000 pound-inches, and the allowable unit-stress was 12,000 pounds per square inch. This would require a section modulus of:

$$S=125{,}000\div 120{,}000=10.50.$$

By reference to column 11 (C 100), the nearest section modulus is found to be 11.2, and this belongs to a 7-in. 17.5-pound I-beam. On (C 102), in column 11, is found a section modulus of 10.5, which belongs to a 9-in. 13-pound channel. In column 9 (C 104) is found the value 11.20, which is the nearest to the required (but must be greater); and this belongs to a 5⅛ by 3⅜ by 13/16-in. Z-bar, which weighs 28.3 pounds per linear foot. Further inspection shows that the 6⅛ by 3⅝ by ½-in. 21.0-pound Z-bar gives an S of 11.22; and since it weighs less, it is cheaper. It is now seen that either a 7-in. 17.5-lb. I-beam, or a 9-in. 13.25-lb. channel, or a 6⅛-in. 21.0-lb. Z-bar will be sufficiently strong to carry the load which produces the bending moment of 126,000 pound-inches. By looking at the weights per foot, it is seen that the channel is the best one to use, and hence the following is true:

When the loads are vertical and the beam is also in a vertical position (that is, standing up on edge, with web or central portion perpendicular), **and when one beam will be sufficient to carry the load, the channel is the most efficient section.**

When the loads are vertical and the beam is inclined (as, for example, when it is on a roof truss, Fig. 44), **the Z-bar is the most economical section,** as can readily be proved by higher mathematics. In this case the section modulus of a beam is different from that given in column 9 (C 104). Since Z-bars are seldom used as

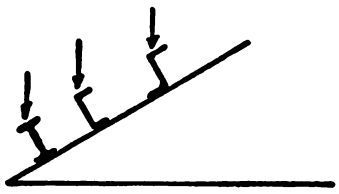


Fig. 44. Classes of Purlins.

beams except in roof work (and then they are placed in position as in Fig. 44), the S for the sizes most used are given in Table XI for the different slants of roof most used. For other slopes, the values will lie between those given, and may be very closely determined from the values given. Thus, suppose the slope of your roof was between 30 degrees and $\frac{1}{3}$—that is, between 30 and 34 degrees. If the slope of your roof was 33 degrees, this would be 33—30=3 degrees above the 30-degree slope; and the section modulus would then be: $8.9-\{(8.9-7.1)\times 3\}\div(34-30)=7.55$ for a $6\frac{1}{8}$-in. 21.0-pound Z-bar.

To show the strength when the Z-bar is turned, the bending moment of 126,000 pound-inches will be taken, and the slope of the roof taken as 30 degrees. The allowable unit-stress being taken as 12,000 pounds per square inch,

the required section modulus is the same as before, 10.5. According to Table X, a 6 1/8 by 3 5/8-in. Z-bar weighing 28.0 pounds per linear foot is required, thus showing a loss of 7 pounds, or an increase of 33 per cent in the cost when compared with the 6⅛-in. 21-pound section as required when it was not tilted.

Channels and I-beams should not be used on roofs, if it is possible to avoid it, since the tilting of these shapes decreases the section modulus very rapidly. If the I-beam and channel as designed were tilted 30 degrees, their strength would be reduced as follows:

Beam	Section Modulus		Moment Capacity
	Vertical	At 30 Degrees	At 30 Degrees
7-in I-Beam	11.2	2.50	18,000 lb.-in.
9-in. Channel	10.5	2.30	15,600 lb.-in.

These section moduli and moment capacities, as compared with the originals, show a decided decrease in strength.

By referring to Fig. 43 it will be noticed that the ends of beams 1, 2, 3, and 4 rest on cross-beams, which are in turn connected with the columns. These cross-beams A-B and C-D are usually termed **floor girders,** even in case they consist of non-built-up beams.

In C-D, the case is that of a beam carrying concentrated loads which are equal to the reactions of the beams 1, 2, 3, and 4. It will be as shown in Fig. 45 the values given being one-half the total loads given on page 180. In the case of

TABLE X

Section Moduli of Z-Bars for Various Slants of Roof

Size	Thickness of Web (Inches)	Weight per Foot (Pounds)	Pitch of Roof Truss, or Degree of Slope of Roof						Angle for Maximum
			1/4 = 22 Deg.	1/3 = 27 Deg.	30 Deg.	1/2 = 34 Deg.	1/1 = 45 Deg.	Maximum	
6 x3½	5/16	15.6	7.8	7.5	6.8	5.1	3.3	9.2	25 Degrees
6 1/16 x3 5/16	3/8	18.3	8.7	9.0	7.7	6.3	4.0	10.7	
6 1/16 x3 3/8	7/16	21.0	9.9	10.5	8.9	7.1	4.6	12.2	
6 x3½	3/8	22.7	10.5	10.7	9.0	7.3	4.7	12.5	
6 1/16 x3 5/16	7/16	25.4	11.6	11.9	10.0	8.4	5.5	14.0	
6 1/16 x3 3/8	1/2	28.0	12.5	12.8	11.0	9.3	6.2	15.3	
6 x3½	7/16	29.3	12.7	13.5	11.1	9.3	6.2	15.4	
6 1/16 x3 5/16	1/2	31.9	13.9	14.2	12.2	10.2	7.0	16.6	
6 1/16 x3 3/8	9/16	34.6	14.7	13.7	13.4	11.5	7.9	17.0	
5 x3¼	5/16	11.6	4.1	5.4	5.1	4.1	2.7	5.9	24 Degrees
5 1/16 x3 5/16	3/8	13.9	5.0	6.4	6.2	5.1	3.3	7.2	
5 1/16 x3 3/8	7/16	16.4	5.8	7.4	7.5	6.0	3.8	8.4	
5 x3¼	7/16	17.9	6.0	7.8	7.6	6.2	4.0	8.7	
5 1/16 x3 5/16	1/2	20.2	6.7	8.7	8.6	7.3	4.6	9.6	
5 1/16 x3 3/8	9/16	22.6	7.0	9.4	9.3	8.0	5.2	10.6	
5 x3¼	11/16	23.7	7.1	9.6	9.5	8.1	5.2	10.8	
5 1/16 x3 5/16	3/4	26.0	7.3	10.3	10.3	8.7	5.8	11.5	
5 1/16 x3 3/8	13/16	28.3	8.5	11.3	11.3	9.7	6.5	12.5	
4 x3 1/16	1/4	8.2	2.1	2.5	3.1	3.6	2.1	3.8	33½ Degrees
4 1/16 x3 5/16	5/16	10.3	2.6	3.2	3.8	4.5	2.7	4.7	
4 1/16 x3 3/8	3/8	12.4	3.0	3.7	4.4	5.5	3.3	5.6	
4 x3 1/16	7/16	13.8	3.2	3.9	4.7	5.6	3.4	5.8	
4 1/16 x3 5/16	1/2	15.8	3.6	4.4	5.2	6.5	4.1	6.6	
4 1/16 x3 3/8	9/16	17.9	4.0	4.9	5.8	7.1	4.7	7.3	
4 x3 1/16	5/8	18.9	4.1	5.0	5.9	7.3	4.7	7.4	
4 1/16 x3 5/16	11/16	20.9	4.4	5.4	6.4	7.8	5.3	8.0	
4 1/16 x3 3/8	3/4	23.0	4.8	5.9	7.0	8.6	5.9	8.8	
3 x2½	1/4	6.7	1.10	1.30	1.50	1.85	1.80	2.45	39 Degrees
3 1/16 x2¾	5/16	8.4	1.35	1.65	1.85	2.25	2.20	3.00	
3 2x2½	5/16	9.7	1.45	1.70	2.00	2.40	2.45	3.25	
3 1/16 x2¾	3/8	11.4	1.65	2.00	2.25	2.70	2.95	3.80	
3 x2½	7/16	12.5	1.70	2.05	2.30	2.85	3.15	3.95	
3 1/16 x2¾	1/2	14.2	1.95	2.35	2.60	3.25	3.60	4.45	

* The pitch of a roof is the value obtained by dividing the rise of the peak by the span.

A-B, the girder not only carries the reactions of beams 1, 2, 3, and 4, but also the reactions of beams 11, 12, 13, and 14. These last-mentioned

Courtesy of American Bridge Co.

ARROTT BUILDING, PITTSBURG, PA., UNDER CONSTRUCTION.

Dimensions, 72 ft. by 82 ft.; 18 stories and attic. Weight of steel used, 3,000,000 lbs.

PLATE 22—STEEL CONSTRUCTION.

Courtesy of American Bridge Co.

JAMES D. PHELAN BUILDING, SAN FRANCISCO, CAL., UNDER CONSTRUCTION.

Weight of steel used, 12,500,000 lbs.

PLATE 23—STEEL CONSTRUCTION.

beams may or may not carry the same floor load as the others; and they may or may not be of the same span. Let the span be 16 feet, and the load

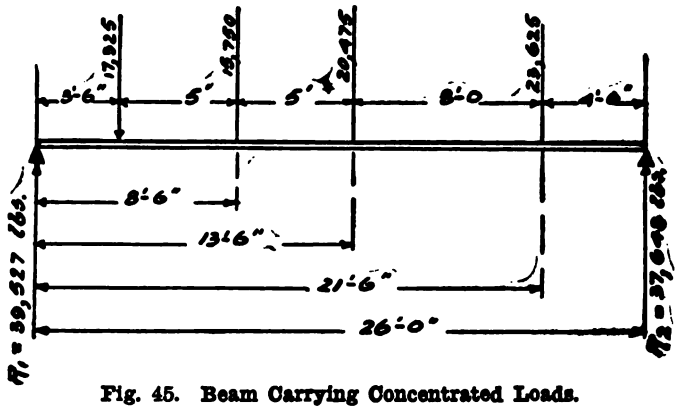


Fig. 45. Beam Carrying Concentrated Loads.

the same as before, 315 pounds per square foot of floor surface. The total loads and reactions are:

Beam	Total Load	Reaction
12	$315\times16\times\{(6+5)\div2\}=27,720$	$27,720\div2=13,860$ lbs.
13	$315\times16\times\{(5\times5)\div2\}=25,200$	$25,200\div2=12,600$ "
14	$315\times16\times\{(5+8)\div2\}=32,760$	$32,760\div2=16,280$ "
11	$315\times16\times\{(8+7)\div2\}=37,800$	$37,800\div2=18,900$ "

The beam A-B will then be as in Fig. 46, a simple beam with concentrated loads equal to the sum of the reactions of the beams that meet at the various points.

The design of beam B-C will now be made. The reaction R_1 is:

$$R_1=\{(23,625\times4\tfrac{1}{2})+20,475\times(4\tfrac{1}{2}+8)+15,750\times(4\tfrac{1}{2}+8+5)+17,325\times(4\tfrac{1}{2}+8+5+5)\}\div26=39,527 \text{ pounds.}$$

The right reaction R_2 is now readily computed to be 37,648. The reader should do this work.

The moments under the several loads are now computed as follows:

At Load		Moment in Pound-Inches
17,325	$39,527\times3\frac{1}{2}\times12=$	1,660,134
15,750	$39,527\times8\frac{1}{2}\times12-17,325\times5\times12=$	2,992,254
20,475	$39,527\times13\frac{1}{2}\times12-(17,325\times10\times12+15,750\times5\times12)=$	3,379,374
23,625	$39,527\times21\frac{1}{2}\times12-(17,325\times18\times12+15,750\times13\times12+20,475\times8\times12)=$	2,033,166

This shows that the greatest moment occurs under the load of 20,475 pounds.

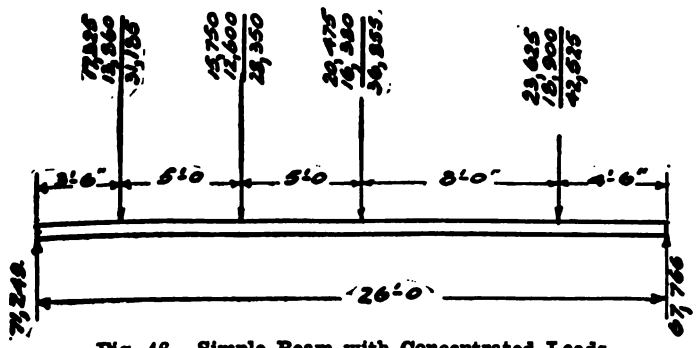


Fig. 46. Simple Beam with Concentrated Loads.

If the allowable unit-stress is 16,000 pounds per square inch, the section modulus is:

$$S=3,379,374\div16,000=211.21$$

By consulting (C 98), column 11, it will be seen that not even the largest I-beam is strong enough. If the loads on the floor were fixed, as

in the case of heavy machinery, a unit-stress of 20,000 pounds per square inch would be allowable. The S required will then be:

$$S=3,379,374\div 20,000=168.97,$$

and a 24-in. 80-pound I-beam may be used. This has S=174.0, but it is the value which comes nearest to the required 168.97.

In case the allowable unit-stress were 16,000 pounds per square inch, as required above, two I-beams riveted together would be used, or, better still, a built-up girder such as is discussed in the next article.

In Fig. 46, where A-B is shown, the reaction will be as shown, and the greatest moment will occur under the 36,855-pound load; and it will be 6,083,219, which, at 16,000 pounds per square inch, requires a section modulus of $6,083,219\div 16,000 = 380.1$; and here, as before, two large I-beams or a built-up girder may be used.

The most difficult case has been discussed in the foregoing presentation. Usually, in practice, few such cases occur in building work. The floor joists or beams are usually spaced equally, and the columns are also spaced so that the cross or floor girders are of the same length through the building (see Fig. 47). This simplifies matters considerably, since all the beams are alike and the cross-girders are loaded symmetrically both as to position and as to weight of loads. If the floor load due to its own weight was 20 pounds per square foot, and that due to the load

it was to carry was 80 pounds per square foot, then the cross-girders of Fig. 47 would be loaded as shown in Fig. 48, each load being $18 \times 4 \times 100 = 7{,}200$ pounds. This, it will be noticed, is the amount of load that comes on the shaded area, Fig. 47. It is twice the reactions of one beam. Each reaction of the cross-girder (see page 163) is equal to $2 \times 7{,}200 = 14{,}400$ pounds.

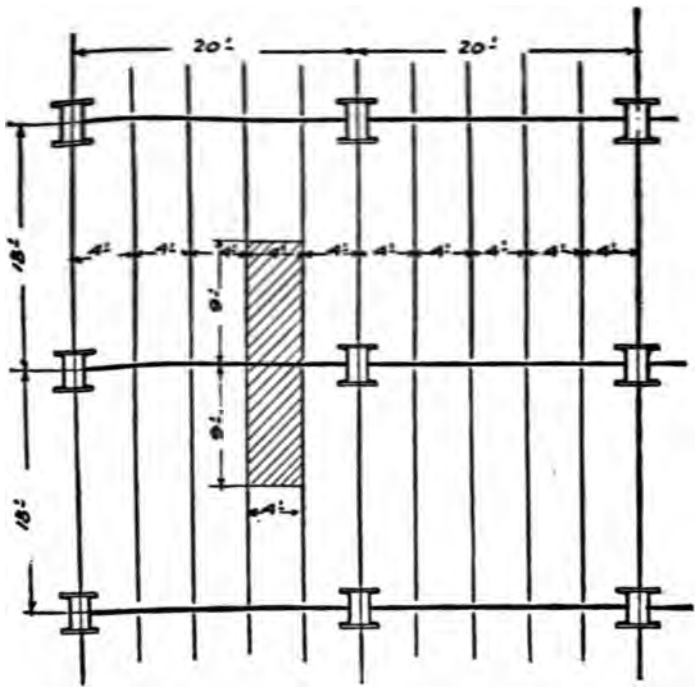


Fig. 47. Framing when Joist Spacing is Equal.

There should always be an uneven number of spaces on the cross-girder, as in Fig. 48, where there are five. If there were an even number, one floor-beam would come in the cen-

ter of the cross-girder (see Fig. 49), and this would (as you will find by designing the beam) require a larger cross-girder to carry the same load than would be required if the number of spaces were uneven. In cases where they are spaced as in Fig. 48, the moment will be the same

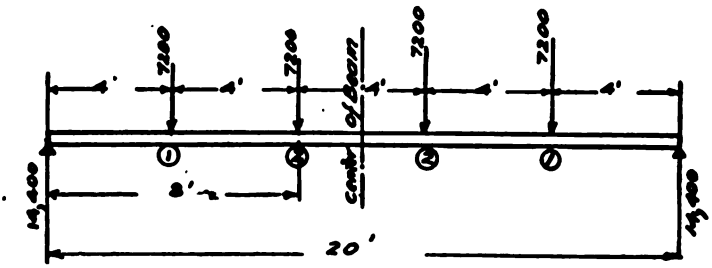


Fig. 48. Floor-Girder—Joist Spacing Equal.

under each of the beams equidistant from the center of the cross-girder; and also it will be the greatest under the beams nearest the center of

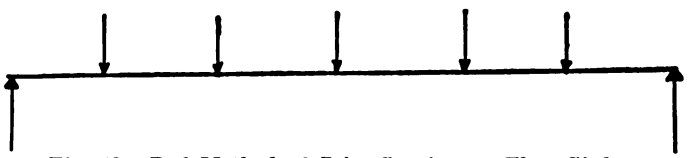


Fig. 49. Bad Method of Joist Spacing on Floor-Girder.

the cross-girder. In the case of Fig. 48, the greatest moment is:

M_2=14,400×8×12–7,200×4×12=1,036,800 pound-inches.

The design of the small beams which carry 7,200 pounds can be made by the second method page 184). Here, if the allowable unit-stress is 16,000 pounds,

C=20×1=20.00;

and in C 100 . 1 the column is found 151.700, the nearest value that can be used. This belongs to an 8-in. 18-pound I-beam.

The cross-girder will be designed by its section modulus. The required S is:

S=1.25×80−11.00=45;

and from column 11 (C 98 . a 15-in. 50-pound I-beam is seen to be just too small. A 15-in. 55-pound I-beam will be used.

In all of the above cases the weights of the beams have been neglected, since they are less than one-tenth of the load they carry.

In practice, where a lot of joists are equally spaced along one cross-girder, it is, by some poor designers, considered that the action of the load is the same as if it were a uniform load over the entire cross-girder. The cross-girder in question will now be so designed by this method.

C=20×4×7.200=576.000

This would call for only a 15-in. 42-pound I-beam, whereas the correct design requires a 15-in. 55-pound beam—a beam 13 pounds heavier. This shows that this practice is incorrect, and should not be followed.

On pages (C 91-94) is much valuable information regarding the tables of beams and their loads. When two or more letters are written together, it means that they are multiplied together. For example **Pl** means that **P** is multi-

plied by **l**. If one set of letters or figures appear with a line between, it means that the top one is divided by the bottom one; as **Pl/AE** means that **P** is multiplied by **l**, and the result is divided by **A** multiplied by **E**. In case a small figure is seen up near the top of the letter, as l^2 or l^3, it means that the number is multiplied by itself that number of times. That is, $l^2=l\times l$, and $l^3=l\times l\times l$. The understanding of the foregoing notation is essential to the person who would use (C 91-94).

In order not to have the **plastering crack** in buildings on account of the bending of the beam, the load must not exceed a certain amount, even if it can carry more. In (C 67-69) is given much information regarding the **limits of span length and loading.** In tables (C 70-74) are horizontal black lines in the columns. This signifies that only beams above that line can be used if plastering is on their under surface. For example, on page 70, looking in the 15-in. 80-pound I-beam column, you will see that this beam can be used for spans from 12 to 24 feet, if plastering is used; and if it is not used, these may be used up to 36 feet, the loads they carry in each case varying with the span.

39. **Shears and Moments in Girders.** These are computed in exactly the same manner as for beams. Unless the load is a uniform one, designing of girders by taking their entire section directly from the Carnegie Handbook is rather limited. The determination of the sizes must

usually be made by design, but in case a "Godfrey's Handbook" is available, girders of almost any size and for any purpose can be taken directly from the tables given therein. This handbook is invaluable to the advanced worker, and is of great advantage to the beginner.

GIRDERS

40. **Description and Classification.** A girder may be designated as a **girder** or a **plate-girder,** and each of these classes are again divided into **simple** and **box** girders. These and all girders may be divided into **deck** and **through** girders. A girder is a beam which consists of a channel, or an I-beam, or any other single rolled shape.

A **plate-girder** is a beam which consists of a plate called a **web plate,** with a combination of angles, or one or more plates and angles, riveted on the outer edges. These are called the **flanges.** The angles are called the **flange angles,** and the plates are called **flange-plates** or **cover-plates.** In case the web is deep, angles are riveted vertically on its sides at intervals along its length. These are called **stiffener angles.** Fig. 50 gives sections of plate-girders; and Fig. 51 gives a side view of a plate-girder with cover-plates. These cover-plates are not always the same length, for reasons that will be explained later. At the ends of one of the plate-girders, are small plates which are placed between the girder and

whatever it rests upon. These are called **masonry plates** or **wall-plates.**

When two girders or plate-girders are placed close together and riveted up, they form a **box girder.** Fig. 52 gives sections of box girders, and (C 152—156, 161, and 162) are tables giving

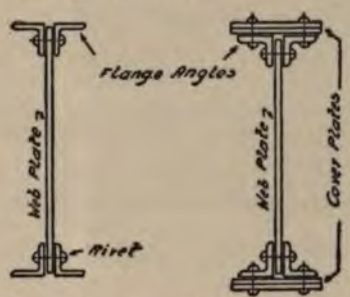


Fig. 50. Sections of Plate-Girders.

the carrying capacity of box girders when loaded uniformly. Box girders are much used in building work to carry walls and elsewhere where uniform loads come upon them; but they are seldom used for concentrated loads, on account of the fact that beams cannot be connected to their webs, as the riveters cannot get tools inside to rivet up the connections, and, if the beams rest on top of them, one side is liable to bend more than the other, and the stress will be far greater than that for which they were designed.

41. **Design of Girders.** Since box girders should be used for uniform loads only, and since the unit-stress (C 152-162) of 15,000 pounds per square inch is good practice, no other

method except that of the handbook will be given here.

Suppose that the distance center to center of columns was 20 feet, and that the girder was to carry an 18-inch brick wall 30 feet high. In (C 58) it is found that such a wall weighs 168 pounds for each square foot of wall surface.

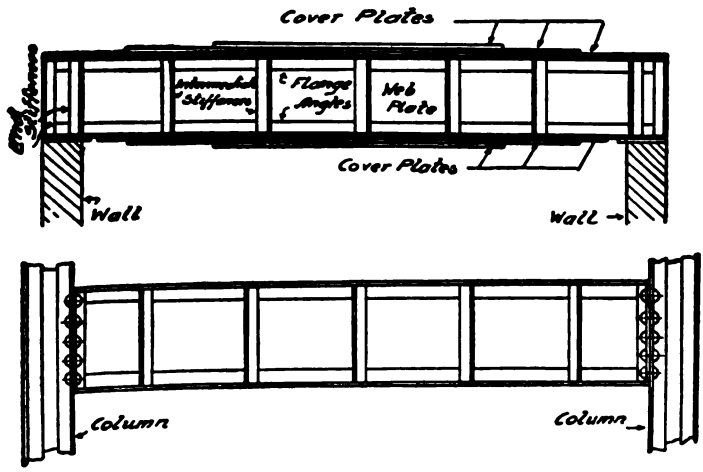


Fig. 51. Wall Bearing and End-Connected Girders.

The total weight carried is, then, $30 \times 20 \times 168 = 100{,}800$ pounds. To reduce this to tons, divide by 2,000, the number of pounds in a ton. This gives $100{,}800 \div 2{,}000 = 50.4$ tons. In (C 154) under the 15-in. 42-pound box girders with two 14-in. by 5/8-in. plates, it is found that for a span of 20 feet the girder will carry 53.04 tons, including its own weight, which, according to the second column, is 1.47 tons. This gives a net carrying capacity of $53.04 - 1.47 = 51.57$ tons, which

is as close as we can get. This beam will be used.

For a good example in design, see (C 151). An inspection of the box plate-girders shows that they are all too heavy for the load and span as given above.

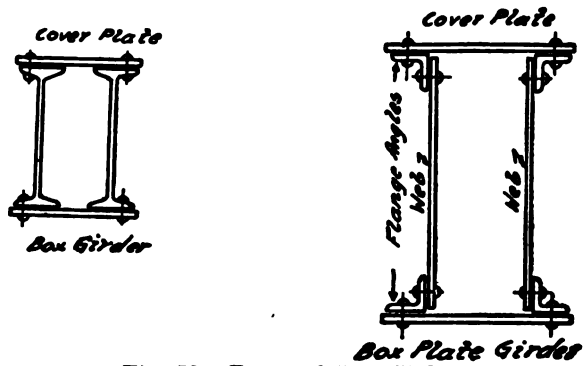


Fig. 52. Types of Box Girders.

For example, suppose that the wall to be carried in a certain case was 381,620 pounds, and that the span was 35 feet. The load is 381,620÷2,000=190.81 tons. On the right-hand side of (C 162), across from the span of 35 feet, is found 188.10 tons in the first column. The girder weighs 6.43 tons, so that the net carrying capacity is 188.10—6.43=181.67 tons. In the third column it is seen that if the flange plates are increased in thickness 1/16th inch, the girder will carry 10.45 tons more, thus making a net capacity of 181.67+10.45=192.12, which, being slightly greater than the required capacity of 190.81, shows the section is all right. The girder section will now be:

2 web plates 42 in. by $\frac{1}{2}$ in.;
4 angles 5 in. by $3\frac{1}{2}$ in. by $\frac{1}{2}$ in.;
2 cover-plates 30 in. by $\frac{3}{4}$ in.

The increasing of the cover-plates 1/16th inch, also, as noted in the extreme right-hand column, increases the weight of the girder 0.22 ton. This is too small to be considered, and should be used only in computing the weight of the girder to see how much it would cost.

The design of plate-girders for unit allowable stresses other than 15,000 pounds per square inch, and for either uniform or concentrated loads, is done by use of the maximum moment which the loads produce.

Thicknesses of flange angles greater than 13/16 inch should be avoided, since it will be necessary to bore the holes instead of punching them, and this increases the cost.

Angles should have the large leg turned out if they are unequal-legged, the small leg being riveted to the web plate.

Web plates should never be thinner than $\frac{3}{8}$ inch, since they would not be stiff enough if they were.

The design of plate-girders may be made by the help of Tables XI and XII.

Rules—

(1) Having the moment due to the loads given, assume a given angle; divide it by the height less the value x, column 4, if a cover-plate is not to be used; and then divide the result by the allowable unit-stress. The

result will be **a**, column 3, and the required angles can be picked out of column 1, Table XI.

(2) Compute the thickness of the web plate; and if this comes out thinner than $\frac{3}{8}$ inch, use $\frac{3}{8}$ inch.

(3) Compute the weight of the girder as now designed.

(4) Compute the bending moment due to its own weight.

(5) Add the bending moment due to its own weight to that due to the loads, and repeat operation (1). The result will be the section of the girder.

If it is desired to have a cover-plate on the girder, assume the sized angle as before, and, using **y** and **b** in place of **x** and **a**, proceed as above.

Cover-plates less than 7/16 inch run the entire length of the girder. If the total thickness of a cover-plate runs up to $\frac{3}{4}$ inch, it should be composed of two plates, one $\frac{3}{8}$ inch thick, and the other $\frac{3}{8}$ inch or less. If the cover-plate runs up to $1\frac{1}{4}$ inches, there should be three cover-plates, each $\frac{3}{8}$ inch thick. In all cases the thinnest plate should be on the outside of the girder. If there are more cover-plates than one, the length may be determined from Table XII.

Rules—

(1) Compute the area of the cover-plates from the top to the under side of the one whose length is desired. This is readily done by multiplying the value in column 8 of Table XI by the number of sixteenths the plate is thick.

(2) Divide the area of the cover-plate so computed by the value **b**, column 6, Table XI, if there be one $\frac{3}{8}$-

TABLE XI

Coefficients for Plate-Girder Flanges

1	2	3	4	5	6	7	8	9
Size Inches	Thickness Inches	Area (a) Sq. Inches	(x) Inches	Cover-Plate	Area (b) Sq. Inches	(y) Inches	Add to (b) for ⅛ in. Increase in thickness of cover-plate	Net Area of Cover-Plate ⅝ in. thick
8 x 8	1⅛ 1⅛ 1 15/16 ⅞ 13/16 ¾ 11/16 ⅝ 9/16 ½	31.20 29.62 28.00 26.36 24.70 22.08 21.38 19.68 17.96 16.24 14.55	4.15	1-18 in. x ⅝ in.	34.74 33.40 32.00 30.48 28.94 27.44 25.88 24.30 22.70 21.12 19.55	3.27	1.000	6.00
6 x 6	1 15/16 ⅞ 13/16 ¾ 11/16 ⅝ 9/16 ½ 7/16 ⅜	20.06 18.86 17.72 16.36 15.38 14.18 12.96 11.74 10.50 9.24 7.96	3.47	1-14 in. x ⅝ in.	22.50 21.48 20.46 19.44 18.38 17.30 16.20 15.12 14.06 12.86 11.70	2.68	0.750	4.50
*6 x 4	1 15/16 ⅞ 13/16 ¾ 11/16 ⅝ 9/16 ½ 7/16 ⅜	16.00 15.12 14.22 12.32 12.38 11.44 10.46 9.50 8.50 7.48 6.46	2.09	1-14 in. x ⅝ in.	20.50 19.62 18.72 17.82 16.88 15.94 14.96 14.00 13.06 11.98 10.96	1.47	0.750	4.50
*6 x 3½	1 15/16 ⅞ 13/16 ¾ 11/16 ⅝ 9/16 ½ 7/16 ⅜	15.00 14.18 13.34 12.50 11.62 10.74 9.84 8.94 8.00 7.06 6.08	1.77	1-14 in. x ⅝ in.	19.50 18.68 17.84 17.00 16.12 15.24 14.34 13.44 12.50 11.56 10.58	1.40	0.750	4.50
*5 x 3½	⅞ 13/16	11.58 10.88	1.83	1-12 in. x ⅝ in.	15.33 14.63	1.21	0.625	3.75

TABLE XI—(Concluded)
Coefficients for Plate-Girder Flanges

1	2	3	4	5	6	7	8	9
Size Inches	Thickness Inches	Area (a) Sq. Inches	(x) Inches	Cover-Plate	Area (b) Sq. Inches	(y) Inches	Add to (b) for ⅛ in. increase in thickness of cover-plate	Net Area of Cover-Plate ⅜ in. thick
* 5 x 3½	¾	10.12	1.83	1-12 in. x ⅜ in.	13.87	1.21	0.625	3.75
	11/16	9.36			13.11			
	⅝	8.58			12.33			
	9/16	7.82			11.57			
	½	7.00			10.75			
	7/16	6.18			9.93			
	⅜	5.34			9.09			
	5/16	4.50			8.25			
* 5 x 3	13/16	10.06	1.47	1-12 in. x ⅜ in.	13.81	0.89	0.625	3.75
	¾	9.38			13.15			
	11/16	8.68			12.43			
	⅝	7.96			11.71			
	9/16	7.24			10.99			
	½	6.50			10.25			
	7/16	5.74			9.49			
	⅜	4.96			8.71			
	5/16	4.19			7.94			
* 4 x 4	13/16	10.06	2.33	1-10 in. x ⅜ in.	13.08	1.60	0.500	3.00
	¾	9.38			52.38			
	11/16	8.68			11.68			
	⅝	7.96			10.96			
	9/16	7.24			10.24			
	½	6.50			9.50			
	7/16	5.74			8.74			
	⅜	4.96			7.96			
	5/16	4.18			7.18			
* 3½ x 3½	13/16	8.44	2.09	1-8 in. x ⅜ in.	10.69	1.47	0.375	2.25
	¾	7.88			10.13			
	11/16	7.30			9.55			
	⅝	6.70			8.95			
	9/16	6.12			8.37			
	½	5.50			7.75			
	7/16	4.86			7.11			
	⅜	4.20			6.45			
	5/16	3.56			5.81			
* 3 x 3	5/8	5.46	1.71	1-8 in. x ⅜ in.	7.71	1.00	0.375	2.25
	9/16	5.00			7.25			
	½	4.50			6.75			
	7/16	3.98			6.23			
	⅜	3.46			5 71			
	5/16	2.94			5.19			
	17	2.38			4.83			
	¼							

TABLE XII

Coefficients for Cover-Plates

Ratio of Cover-Plate Area to Total Area										
Column.....	1	2	3	4	5	6	7	8	9	10
Ratio A....	0.10	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.55
Coefficient Q.....	0.32	0.39	0.45	0.50	0.55	0.59	0.64	0.67	0.71	0.75

inch only. If the plate is thicker than $\frac{3}{8}$ inch, then divide the area of the cover-plate as computed above, by the value given in column 6, after the additional amount as per column 8 has been added to it.

(3) In Table XII find the ratio **A** which is equal to the value just obtained, or next above it in value, and just below will be found the coefficient **Q**.

(4) Multiply the span of the girder by **Q**, and the result will be the length of the cover-plate on the bottom.

One thing should be avoided. No section should be used for a flange in which **b**, column 6, Table XI, is over twice as great as the value **a**, but it shows good designing to have it almost twice as great as **a**.

Angles marked with a * in Table XI have one $\frac{7}{8}$-inch rivet-hole out of the section of each angle; and when a cover-plate is used, two $\frac{7}{8}$-inch rivet-holes are out of this also (see Fig. 53). Angles not marked have one rivet-hole taken out of each angle; and when a cover-plate is used, two rivet-holes are taken out of each angle, and two out of the cover-plate (see Fig. 54). It will be noticed in Figs. 53 and 54, that the web plate does not extend up to the back of the angles. This is done in order that, if any

SHOWING USE OF GALLOWS FRAME AND DERRICK CAR IN PLACING A PLATE-GIRDER.

SHOWING USE OF DERRICK CAR RUNNING ON TRESTLE WORK.

PLATE 24—STEEL CONSTRUCTION.

LOCOMOTIVE CRANE ERECTING A 105-FT. 6-IN. PLATE-GIRDER.

Over Passaic River, at Lyndhurst, on line of Delaware, Lackawanna & Western Railroad.

PLATE 25—STEEL CONSTRUCTION.

little irregularities occur in the rolling, they will not stick out above the backs of the angle. This distance is usually made $\frac{1}{8}$ inch in practice, and

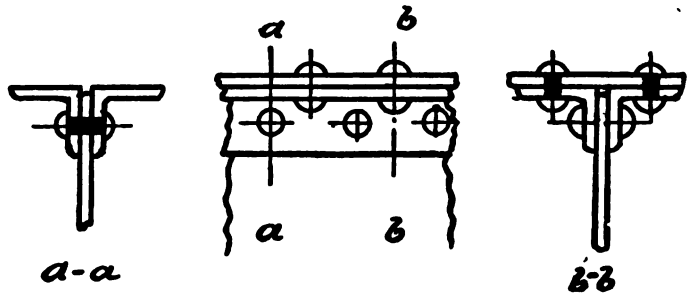


Fig. 53. Rivets Out with Single Gauge Line.

Table XI is figured on that basis. From Fig. 53 it is seen that the rivets in the cover-plates must not come opposite those in the leg of the angle which lies against the web, while in the angle of Fig. 54 this may be done.

In Table XI the value **a** is the area of the two angles after the two rivet-holes have been

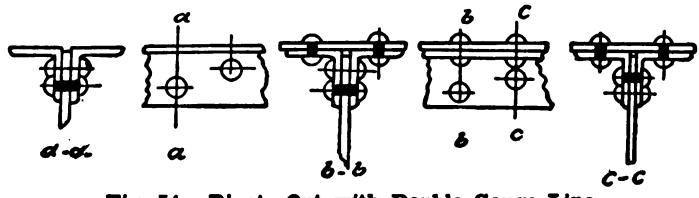


Fig. 54. Rivets Out with Double Gauge Line.

subtracted; and **b** is the area of the cover-plate and angles after rivet-holes have been taken out as per Figs. 53 and 54.

The values **x** and **y** are the distances from the edge of the web plate to the center of gravity

axis of the angle or angles and plates. These values vary slightly with the thickness of the angles, and also with the thickness of the cover-plate; but for practical purposes are constant as shown. The **center of gravity** of a pair of angles, or of two angles and a cover-plate, is that point at which, if they were supported, they would just balance (see Fig. 55). And the **center of gravity axis** is a line passing through this point and parallel to the other leg of the angles, as shown in Fig. 55.

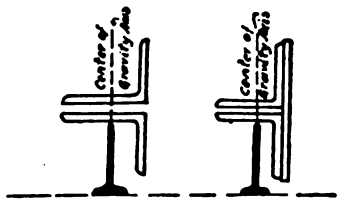


Fig. 55. Illustrating Location of Center of Gravity Axis.

As an example of the use of the tables, let it be required to design the section of a plate-girder which has previously been computed to have a bending moment of 5,330,574 pound-inches, the span being 40 feet, and the unit allowable stress being taken as 15,000 pounds per square inch. If the girder is for railroad or highway use, the depth will usually be stated in the specifications as 1/9 or 1/10 the span or some other ratio. In building operations the depth is usually limited by the head-room required in the floor below, or by some other condition of construction.

Suppose this was a building girder, and the depth was limited to 40 inches. Assume a 5 by 3½-in. angle to be used. The assuming of this angle is a matter of experience. A good guide, however, is to divide the bending moment as given by the allowable unit-stress, and then divide this by the depth chosen, less two inches. The result will then be somewhere near the required area **a** or **b** of Table XI. In this case,

$$(5{,}330{,}574 \div 15{,}000) \div (40-2) = 9.35.$$

From Table XI it is seen that perhaps either a pair of 5 by 3½ by ¾-in. angles or two 5 by 3½ by 7/16-in. angles, with a 12 by ⅜-in. cover-plate, may be used. Two 5 by 3½ by 11/16-in. angles give an area very close, but the weight of the girder will increase the above moment, and hence the value **a** or **b**. This computation shows that the value can fall in the 5 by 3½-in. angles.

A design will first be made without a cover-plate. Then,

$$a = (5{,}330{,}574 \div 15{,}000) \div (40-1.83) = 9.31 \text{ square inches.}$$

When the weight of the girder is considered, the required area will be greater than this. For the purpose of making an estimate of weight, the angles which give areas next greater than this value are to be used. They are 5 by 3½ by ¾-in., and weigh, (C 111), column 4, 19.8 pounds per linear foot for each angle, or $2 \times 19.8 = 39.6$ pounds per linear foot for each flange. Since there are two flanges, these weigh $2 \times 39.6 = 79.2$

pounds per linear foot of girder. The girder is 40 feet long; and therefore the weight due to the flanges is $79.2 \times 40 = 3{,}168.0$ pounds. To this must be added the weight of the web plate. The web plate is designed in a manner similar to that given on page 188. It will be supposed that the greatest reaction is 22,210 pounds, which would be the case if one load of 44,420 pounds was at the center of the span, causing the moment of 5,330,574 pound-inches. The area required in the web is then:

$$\text{Area} = \frac{22{,}210}{10{,}000} = 2.22 \text{ square inches,}$$

the allowable unit-stress being taken as 10,000 pounds per square inch. The required thickness is:

$$t = \frac{2.22}{40} = 0.055 \text{ inch.}$$

The web will be made $\frac{3}{8}$ inch thick, as explained on page 204. The web is now a plate 40 inches wide and $\frac{3}{8}$ inch thick. From (C 259) the weight given in the 40-inch column and on the $\frac{3}{8}$-inch line is 51.00 pounds per linear foot. The total weight of the web is then $51.0 \times 40 = 2{,}040$ pounds. The weight of the flanges and the web is:

Flanges,	3,168	pounds;
Web	2,040	"
Total	5,108	pounds.

The weight of the rivet-heads and the stiffener

angles amounts to about 1/10 of this, or 5,108÷10=511 pounds. The total weight of the girder is now found to be 5,108+511=5,619 pounds; and the maximum moment due to this weight (see page 175) is:

$$M=\frac{5{,}619\times40\times12}{8}=337{,}140 \text{ pound-inches.}$$

This must now be added to the amount caused by the loads, thus giving a total of 5,330,574+337,140=5,667,714.

The area **a** is now computed as follows:

$$a=(5{,}667{,}714\div15{,}000)\div(40-1.83)=9.899 \text{ square inches,}$$

which shows, from Table XI, that two 5 by 3½ by ¾-inch angles, giving an area of 10.12 inches for **a**, are the ones for the flange. The section of the girder is shown in Fig. 56.

From an inspection of column 6 of Table XI, it appears as if two 5 by 3½ by 7/16-inch angles, with a cover-plate 12 by ⅜ inch, may be better; or perhaps two 5 by 3½ by ⅜-inch angles, with a cover-plate thicker than ⅜ inch, may do. This will now be investigated.

The weight of the girder with a cover-plate will be taken the same. The small difference will not affect the stresses enough to make any difference in the design. The required area is:

$$a=(5{,}667{,}714\div15{,}000)\div(40-1.83)=9.899 \text{ square inches,}$$

This shows that two 5 by 3½ by 7/16-inch angles and one cover-plate 12 by ⅜-inch can be used,

the total net area being 9.93 square inches. The web will be the same size. This gives a lighter flange area than that designed without a cover-plate.

As a general thing it is better practice to

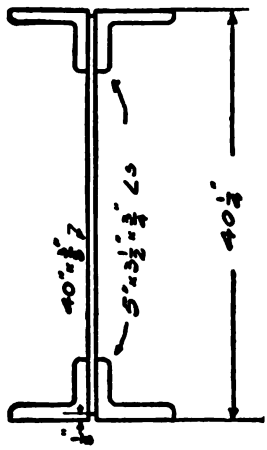


Fig. 56. Section of Plate-Girder.

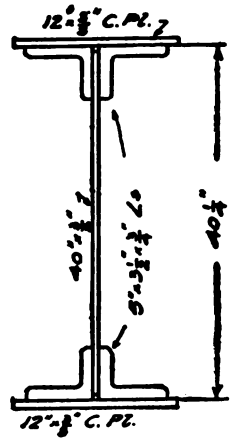


Fig. 57. Section of Plate-Girder with Cover-Plates in Flanges.

design with a cover-plate, as this not only gives less flange area, but the cover-plates join the flanges together better, and a more even distribution of stress is assured. The cover-plate will extend the full length, and will be on both top and bottom, of course, since both flanges should be made the same.

Fig. 57 gives a section of the girder as designed with a cover-plate. The weight of the flanges and web per linear foot are:

Flange:		
	2 Cover-Plates (C 256), 15.30×2=	30.60
	4 Angles 5 by 3½ by 7/16 in. (C 111), 12.00×4=	48.00
Web:		
	1 Plate 40×⅜=	51.00
Weight of girder per linear foot,		129.60 pounds

The design, without cover-plate, weighed 130.2 pounds per linear foot, thus showing a small saving in weight. The extra work necessary to rivet on the cover-plate makes cover-plate sections more costly; but the design is better on account of the better riveting together of the outstanding legs of the angles.

The thickness of the end stiffeners of a plate-girder should be:

For reactions up to 225,000 pounds.....	⅜ Inch
For reactions from 225,000 to 250,000 pounds....	7/16 Inch
For reactions from 250,000 to 275,000 pounds....	½ Inch
For reactions from 275,000 to 300,000 pounds....	9/16 Inch
For reactions from 300,000 to 400,000 pounds....	⅝ Inch
For reactions over 400,000 pounds.....	¾ Inch

The intermediate stiffeners should always be at least ⅜ inch in thickness. The size of stiffeners should be such that they will extend out as far as possible, and thus support the horizontal leg of the flange angles; and the leg on the web should be such as to get one row of rivets in it. This leg last-mentioned should be from 3 inches to 4 inches. By consulting (C 13-14), it will be noticed that the angles are curved off at the ends, and that this curvature is never greater

than $\frac{3}{8}$ inch in any case. From inspection of Fig. 58, it is seen that the outstanding leg of a stiffener cannot be more than:

$$f=l-(t-\tfrac{3}{8}\text{ inch}),$$

in which l is the outstanding leg of the flange angle; t is the thickness of the flange angle; and $\frac{3}{8}$ inch is the curvature as above explained.

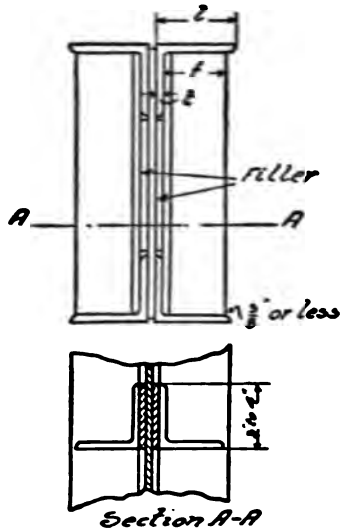


Fig. 58. Showing Length of Outstanding Leg of Filler.

For example, suppose that the flange angles were 6 in. by 4 in. by ½ in., the 6-inch leg being horizontal. Here l=6, and t=½; then the stiffener leg cannot be greater than:

$$f=6-(\tfrac{1}{2}+\tfrac{3}{8})=5\tfrac{1}{8}\text{ inches.}$$

Therefore we can make it 5 inches, and the stiffener angle will be 5 by 3 inches or 5 by 4

inches in size, the thickness depending upon whether it is an end or an intermediate stiffener.

A girder requiring several cover-plates will now be designed in order to show how the cover-plates vary. Let the span be 30 feet, the depth of the web-plate 50 inches, and the loads so fixed that the maximum moment is 10,245,900 pound-inches. The allowable unit-stress will be taken as 10,000 pounds per square inch. In order to determine what sized angles will be used, an approximate **b** will be computed as is done on page 211. It is:

$$\text{Approximate } b=\frac{10{,}245{,}900}{10{,}000+(50-2)}$$

$$=\frac{10{,}245{,}900}{10{,}000\times 48}$$

$$=21.3 \text{ square inches.}$$

By looking at the 5 by 3½-inch angles and cover-plates of Table XI, it will be seen that if three ⅜-inch cover-plates are used, and two 5 by 3½ by ¾-inch angles, the area will be $13.87+2\times 3.75=21.37$ square inches. The 5 by 3½-inch angles will be tried.

$$b=(10{,}245{,}900\div 10{,}000)\div(50-1.21)$$

$$=21.00 \text{ square inches required.}$$

If 5 by 3½ by 4-inch angles are used with one cover-plate, the area would be 13.87 square inches. Therefore thicker cover-plates are required. The amount to be made up by the greater thickness of the cover-plates is $21.00-13.87=7.13$ square inches. As $^1/_{16}$ inch gives

an increase of 0.625 square inch, there will be required 7.13÷0.625=11.4 sixteenths—or $^{12}/_{16}$, as a fraction of a sixteenth cannot be used. This will make $^{12}/_{16}$ inch, or $\frac{3}{4}$ inch, to be added to the thickness of the cover-plates.

It will be assumed that the weight of the girder is less than $^{1}/_{10}$ of the load it carries, and that the specifications will in this case allow the weight to be neglected; otherwise the weight

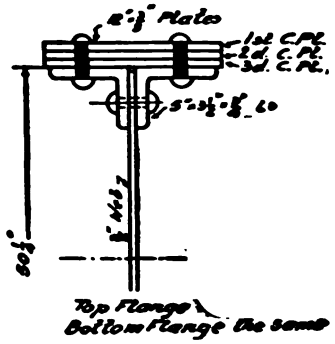


Fig. 59. Showing Location of Cover-Plates.

must be computed in the same manner as is done on page 211, and the section re-designed. This would only increase the thickness of the cover-plates.

The section is as shown in Fig. 59, the web being $\frac{3}{8}$ inch thick. It may be said that unless the depth of the girder is less than $^{1}/_{12}$ of the span, a $\frac{3}{8}$-inch web will be sufficient. The total net area of the flange is:

3 Cover-plates, 12 by $\frac{3}{8}$ in., at 3.75 each=11.25 sq. in.
2 Angles 5 by $3\frac{1}{2}$ by $\frac{3}{4}$ in., at 5.06 each=10.12 sq. in.
Total 21.37 sq. in.

The area of the first cover-plate is 3.75 square inches, and the coefficient is 3.75÷21.37=0.175, say 0.18. This lies about half-way between the ratio values in columns 2 and 3. Therefore the coefficient value will lie between the coefficient values in columns 2 and 3. The difference between the ratios is 0.20-0.15=0.05, and the difference between the coefficient is 0.45-0.39=0.06. Now, the difference between the ratio 0.15 and the required ratio is 0.18-0.15=0.03. Therefore the required coefficient is:

$$Q=0.39+0.06\times\frac{0.03}{0.05}$$
$$=0.39+0.036 \text{ (say } 0.39+0.04)$$
$$=0.43;$$

and the length of the cover-plate is this value, times the length of the span; or:

Length of first ⅜-in. cover-plate=0.43×30=12.90 feet.

The second cover-plate (see Fig. 59) has the same area as the first one. The total area from the bottom of plate 2 to the top of plate 1 is 3.75+3.75=7.50 (see p. 205 for rule); and the ratio for this plate length is: 7.50÷21.37=0.35. This requires, 6th column, a coefficient of 0.59; and the length of the second cover-plate is:

Length=0.59×30=17.70 feet.

In a similar manner, the area of the cover-plates above the bottom of the third cover-plate is computed to be 10.25 sq. in.; the ratio is computed to be 0.48; the coefficient is computed to be 0.69; and the length of the cover-plate is 20.7 feet.

In case the moment was caused by a uniform load, or by a load at the middle, the cover-plates are arranged as shown in Fig. 60.

In case the moment is caused by concentrated loads placed certain distances each side of the center, the cover-plates must be placed as in Fig. 61, in order that the girder may be safe. The computing of the actual theoretical lengths for such cases is not within the scope of this work; but the method as shown will not be

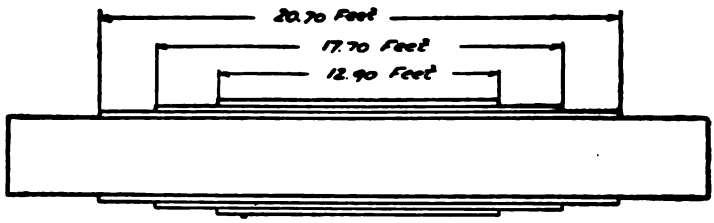


Fig. 60. Symmetrical Cover-Plates.

far from right, and it will be on the side of safety.

From inspection of Figs. 60 and 61, the rule is seen to be that for any cover-plate the length as computed must extend half on each side of the point at which the maximum moment occurs. In case the point of maximum moment occurs closer to the end of the beam than one-half the computed length of one of the cover-plates, that cover-plate extends from the point in question to the end. If there are two points of maximum moment, or two points at which the moment is nearly equal, then the cover-plates extend from each point toward the end

an amount equal to half the length as computed; and they also extend between the loads (see **b**, Fig. 61).

42. **Rivets in Girders.** The girder may be joined to a column at the end, in which case a

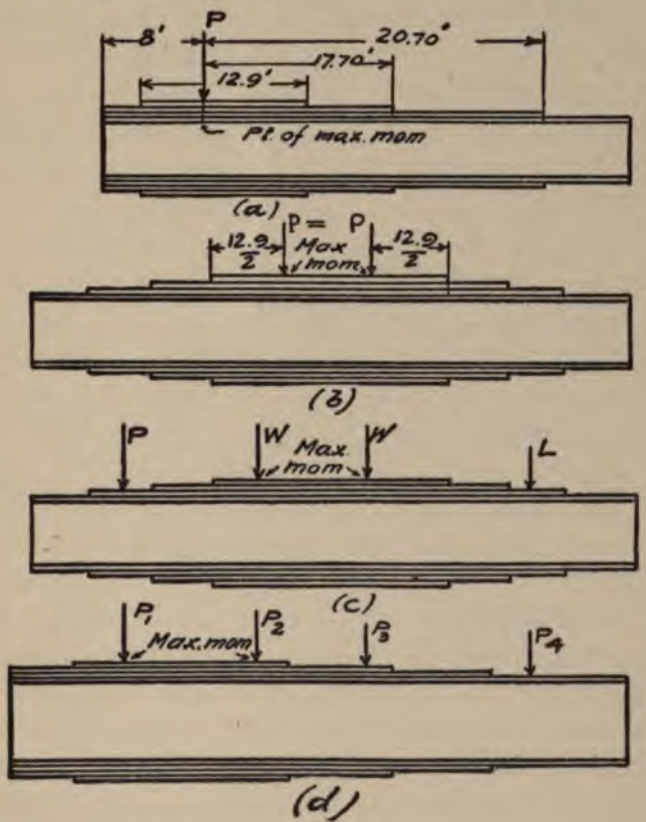


Fig. 61. Unsymmetrical Cover-Plates.

certain number of rivets are required. This number depends upon the unit-stress used, and

upon the thickness of the material to which the column is connected.

In (C 195) are given the bearing values of rivets for a wide range of allowable stresses. These values are for plates of different thicknesses, and the thicknesses are given in fractions of an inch. It is often desirable, as when a girder is joined to a channel or an I-beam, that the bearing value of a rivet in the web of such a shape be known. Table XIII gives data from which this value may be computed for ¾-inch and ⅞-inch rivets.

TABLE XIII

Bearing Values of Rivets

Allowable Stresses	12,000 Lbs. per Sq. In.		15,000 Lbs. per Sq. In.		20,000 Lbs. per Sq. In.		25,000 Lbs. per Sq. In.	
Diameter of Rivet	Value for 0.20-inch Plate	Add for each 0.01 inch	Value for 0.20-inch Plate	Add for each 0.01 inch	Value for 0.20-inch Plate	Add for each 0.01 inch	Value for 0.20-inch Plate	Add for each 0.01 inch
¾ In.	1,800	90	2,248	112	3,000	150	3,752	188
⅞ In.	2,105	105	2,624	136	3,504	175	4,376	219

As an example of the use of Table XIII, let it be required to determine the bearing value of a ⅞-inch rivet in the web of a 12-inch 25-pound channel, the allowable unit-stress for the rivet being 12,000 pounds. The thickness of the channel web is (C 101) 0.39 inch. For a 0.20-inch plate the value is 2,105 pounds; and since the difference between the 0.20-inch plate and the 0.39-inch channel web is 0.39—0.20= 0.19, the value of the rivet will be:

$$v=2,105+19\times105=4,100 \text{ pounds.}$$

Again, with same data as above, except that the channel is an 8-inch 13.75-pound, the thickness of the web is 0.307 inch, and the difference between this and the 0.20-inch plate is 0.307—0.20=0.107 inch. Therefore the value of the rivet is:

$$v=2,105+10.7\times105=3,229 \text{ pounds.}$$

As explained on page 131, the rivet may fail by shear.

Rule—When the ends of a girder or beam are riveted to a plate or column of some form, (1) determine the bearing value of the rivet in the plate or web of column, and (2) compare it with the shearing value of a rivet (C 195). Divide the reaction at that end by the smallest of these values, and the result will be the number of rivets required if the riveting is done **in the shop.** If it is done **in the field,** add one-third more, and the result will be the required number.

Example—Reaction of girder is 87,500 pounds. The rivets are $\frac{7}{8}$ inch in diameter. The allowable unit-stresses are: Bearing 20,000; shear 10,000. The column is composed of two 10-inch 15-pound channels. The web of these channels is 0.24 inch thick. From Table XIII, the bearing value of a $\frac{7}{8}$-inch rivet is

$$V_b=3,504+175(24-20)=4,204 \text{ pounds.}$$

The shearing value (see column 4, table 3, C 195)

is 6,010 pounds. Since the former is the smallest value, the number of shop rivets required would be:

$$n_s=(87,500)\div 4,204=21;$$

and the number of field rivets required would be:

$$n_f=21\times 1\tfrac{1}{3}=28.$$

If, instead of the girder being connected to the column webs, it were connected to a plate $\frac{3}{8}$ inch thick, the rivets would be computed as follows:

v_b in a $\frac{3}{8}$-inch plate (C 195, 3d table)=6,570
v in shear (C 195, 3d table)=6,010

Since the shear is the smallest, the rivets will tend to be cut off instead of tearing out of the

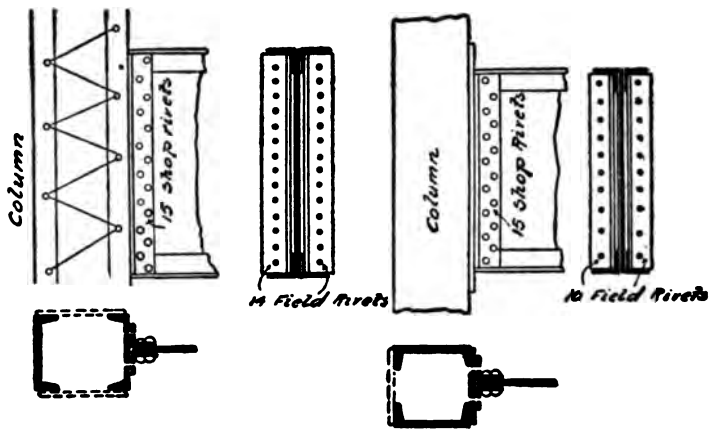


Fig. 62. Methods of Connecting to Columns.

plate, and the number required is 87,500÷6,010 =15 shop; or $15\times 1\frac{1}{3}$=20 field.

More field than shop rivets are required to

Courtesy of "Engineering Record."

CRANE USED IN PLACING GIRDERS IN RAILROAD WORK.

PLATE 26—STEEL CONSTRUCTION.

hold the same force, since it is supposed that a rivet driven by pneumatic or hand tools is not so strong as one driven by the large power riveters in the shops. Fig. 62 shows the connection and where the rivets should go. Field rivets are shown, as such connections are usually field connections.

The rivets as above computed are those which go through the column and the connection angles of the girder. The number of rivets which go through the girder itself and the connection angles must also be determined. Since all webs are $\frac{3}{8}$ inch thick, except in rare cases, this will be the only case treated.

The number of shop rivets is equal to the shear at that end (that is, the end reaction), divided by the value of a rivet in single shear. This riveting is seldom made other than in the shop; but, in case it is done in the field, the field rivets are determined as before. In the case just treated, the required number is $87{,}500 \div 6{,}010 = 15$ (shop); $15 \times 1\frac{1}{3} = 20$ field.

Fig. 62 shows the location, number, and character of these rivets.

In case all the rivets cannot be placed in one row without crowding the spacing too much, the angle should have a leg wide enough to let them be **staggered** in two rows. The flanges must be riveted to the web.

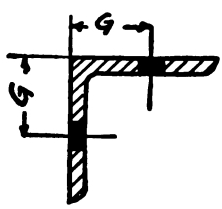
Rivet-Spacing in Girder Flanges. The rivet-spacing depends upon the **shear**, upon the **thickness of the web**, upon the **value of a rivet**, and

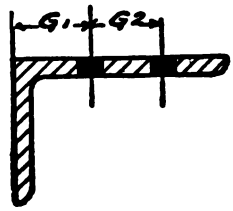
upon the **distance between the rivet-lines of the two flanges.**

The last condition depends upon the distance of the rivets from the back of the angles. All rivets are spaced at a certain distance from the back of the angles, and this distance is standard for a given size of angle. The distance is called the **gauge** of the angle. When an angle can have two lines of rivets in it, that angle is said to have a **double gauge.** The line on which they are placed is called the **gauge-line.** In double gauges, the second gauge-line is measured from the first one, instead of from the back of the angle. Table XIV gives the standard gauges for any sized angle leg.

TABLE XIV

Gauges of Angles





Size of Leg	G	Largest Rivet	Size of Leg	G	Largest Rivet	Size of Leg	G_1	G_2	Largest Rivet
8 in.*	4½ in.	⅞ in.	2¾ in.	1⅝ in.	¾ in.	8 in.	8 in.	8 in.	⅞ in.
7 in.*	4 in.	⅞ in.	2½ in.	1⅝ in.	⅞ in.	7 in.	2½ in.	8 in.	⅞ in.
6 in.*	3½ in.	⅞ in.	2¼ in.	1¼ in.	⅝ in.	6 in.	2¼ in.	2¼ in.	⅞ in.
5 in.	3 in.	⅞ in.	2 in.	1⅛ in.	½ in.	6 in.	2½ in.	2¼ in.	⅞ in.
4 in.	2¼ in.	⅞ in.	1¾ in.	1 in.	½ in.	5 in.	2 in.	1¾ in.	⅞ in.
3½ in.	2 in.	⅞ in.	1½ in.	⅞ in.	⅜ in.				
3 in.	1¾ in.	⅞ in.	1¼ in.	¾ in.	⅜ in.				

*Single gauges are seldom used in this size leg.

In all flanges except those which have a uni-

form load distributed over them (for example, a girder which has a railroad track running over the top), the rivet-spacing at any point is:

$$s = \frac{vh}{V}$$

This means that the spacing is equal to the value of a rivet multiplied by the distance between gauge-lines, and the result is then divided by the shear at that point. All dimensions are in inches.

In cases where beams rest upon the top of a girder, stiffeners should be placed at that point.

In case of railroad girders, as described above, compute the spacing by the given formula, and reduce it ½ inch. This is not the technically exact method; but since it is on the safe side, and the exact method of calculation will involve complicated computations in higher mathematics, the use of this formula will be found perfectly practicable.

In the above formula, **v** is the value of a rivet in bearing in the web of the girder for all thicknesses of web up to ¾ inch inclusive; in webs thicker than ¾ inch, it is the double shearing value of the rivet.

In Fig. 63 is shown a girder with reactions of beams at four places along its length. If the third table (C 195) is used, what will be the rivet-spacing? The shears are computed, and are:

From (0) to (1)=20,000 pounds
From (1) to (2)=20,000–10,000=10,000 pounds
From (2) to (3)=20,000–(10,000+10,000)=0 pounds

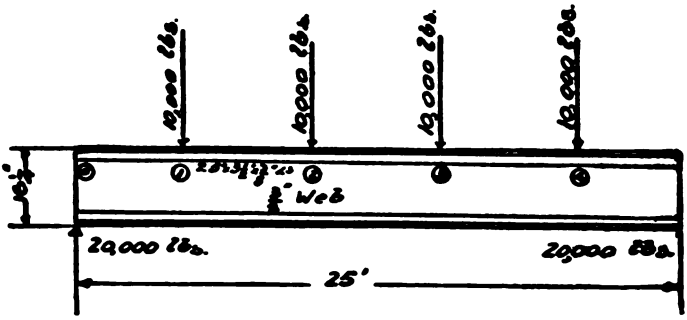


Fig. 63. Illustrating Rivet-Spacing in a Girder with Beam Reactions.

The value of **v**, the web being 3/8-inch, is 6,570 pounds. We compute **h** (see Table XIV) as follows: $h=16\frac{1}{4}-2\times2=12\frac{1}{4}$ inches. Therefore,

From (0) to (1), $s=\frac{6{,}570\times12\frac{1}{4}}{20{,}000}=3.9$, say $3\frac{3}{4}$ inches.

From (1) to (2), $s=\frac{6{,}570\times12\frac{1}{4}}{10{,}000}=7.8$, say $7\frac{3}{4}$ inches.

From (2) to (3), $s=\frac{6{,}570\times12\frac{1}{4}}{0}$=as large as imaginable.

The spacing of $3\frac{3}{4}$ inches would be used; but that of $7\frac{3}{4}$ inches would probably be cut down to 6 inches, since most specifications require this, because it is believed that if the rivet-spacing is larger than 6 inches, it will not hold the angles close to the web and take the kinks out of the metal. Between (2) and (3), the spacing will be 6 inches for the same reason. Of course

the spacing between (3) and (4) is the same as between (1) and (2). Likewise that from (4) to the end is the same as from (2) to the end.

Here the shear between the various loads is constant. In a uniformly loaded beam, this is not so, the shear being different at every point from the end to the center, where it is zero. In such cases, the shear is usually computed at certain points (called **tenth-points**), at distances apart equal to **one-tenth the span**; and in between any two of these points, the spacing is made the same as at the tenth-point to the left.

As an example, let it be required to determine the rivet-spacing at the end and at points separated a distance equal to one-tenth the deck plate-girder span of Fig. 64. This would require the spacing to be found at the end, and at 3, 6, 9, 12, and 15 feet from the end. Of course the rivet-

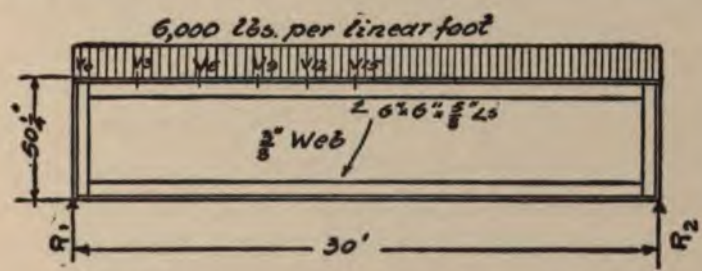


Fig. 64. Illustrating Rivet-Spacing at Tenth-Points.

spacing on the right side of the middle of the girder is the same as that as computed for the left side. The reaction is:

$$R_1=(6{,}000\times30)\div2=90{,}000 \text{ pounds.}$$

The shears (see page 172 for method of computation) are:

$V_0 = 90{,}000 - 0 = 90{,}000$ pounds.
$V_3 = 90{,}000 - 3\times6{,}000 = 72{,}000$ pounds.
$V_6 = 90{,}000 - 6\times6{,}000 = 54{,}000$ pounds.
$V_9 = 90{,}000 - 9\times6{,}000 = 36{,}000$ pounds.
$V_{12} = 90{,}000 - 12\times6{,}000 = 18{,}000$ pounds.
$V_{15} = 90{,}000 - 15\times6{,}000 = 0$ pounds.

If the allowable unit-stress in bearing is taken as 12,000 pounds per square inch, the value of a $\frac{7}{8}$-inch rivet in a $\frac{3}{8}$-inch web is (C 195) 3,940 pounds. The rivet-spacings computed by means of the formula $s = \frac{vh}{V}$, are:

$s_0 = (3{,}940\times43.25) \div 90{,}000 = 1.89$ in., say $1\frac{3}{4}$ in.
$s_3 = (3{,}940\times43.25) \div 72{,}000 = 2.37$ in., say $2\frac{1}{4}$ in.
$s_6 = (3{,}940\times43.25) \div 54{,}000 = 3.16$ in., say 3 in.
$s_9 = (3{,}940\times43.25) \div 36{,}000 = 4.73$ in., say $4\frac{3}{4}$ in.
$s_{12} = (3{,}940\times43.25) \div 18{,}000 = 9.47$ in., say $9\frac{1}{2}$ in.
$s_{15} = (3{,}940\times43.25) \div 0 =$ infinitely great.

This spacing should be decreased $\frac{1}{2}$ inch, since the load rests directly upon the flanges (see page 227). The spacing used would be:

From end to 3-foot section, use $1\frac{1}{4}$-inch spacing
From 3 to 6-foot section, use $1\frac{3}{4}$-inch spacing
From 6 to 9-foot section, use $2\frac{1}{2}$-inch spacing
From 9 to 12-foot section, use $4\frac{1}{4}$-inch spacing
From 12 to 15-foot section, use 6 -inch spacing

43. **Engine and Equivalent Loads.** For any particular plate-girder span, the maximum shear and maximum moments due to any of the stand-

ard locomotives may be obtained from tables given in the specifications for railroad bridges. In order to determine the stresses in bridges under locomotive (wheel) loads, the actual computations must be performed for each particular bridge; and different computations are required for the several members of any given bridge, since the locomotive must be in different positions to give the greatest stresses in different members.

It is much easier to work out the stresses in a bridge under uniform load than under locomotive loads, and so most specifications give what are known as **equivalent uniform loads.** These are the uniform loads which will give the same shear and same moment (as far as practical use is concerned) as the locomotive for which they are given. These equivalent loads are then put over the bridge, and the stresses readily determined. This matter will be more fully explained in our treatment of Stresses in Bridge Trusses.

CLASSES OF COMPRESSION MEMBERS

44. Compression members are those which are stressed by loads or forces at their ends, and these forces act towards each other. If a man pushes on his cane, which has the other end resting upon something, the cane is said to be **in compression,** and it is a **compression member.** This is the direct opposite of the **tension**

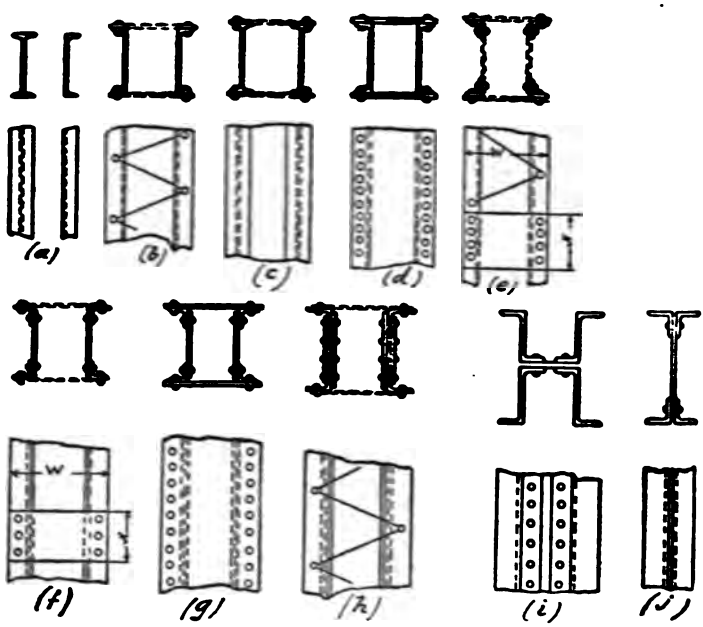


Fig. 65. Sections of Compression Members.

member, which is illustrated by a string which has a pull exerted at each end.

Compression members may be made of different shapes or combinations of shapes. They may consist of single shapes such as a single angle, I-beam, channel, Z-bar, etc., or of two or more angles, two or more Z-bars, or two channels or two I-beams, each combination with or without a plate connecting them. In case plates are not used, the shapes are connected with tie-plates, or with lacing bars and tie-plates. Fig. 65 shows some of the different classes of compression members.

45. **Tie-Plates and Lacing Bars.** The size

of these is not determined by design, but rather by common practice. The size of the tie-plates, often called **batten plates,** is usually limited by the specification, but it is good practice never to have the distance **x** (Fig 65, **e**) less than **w** when the plates are at the end of the member. When there is no lacing in use, the shapes being held together by plates as in Fig. 65, **f**, the distance **x** may be one-half of **w**. With the better class of designers, it is now becoming customary to make **x** (Fig. 65, **e**) $1\frac{1}{2}$ times **w**.

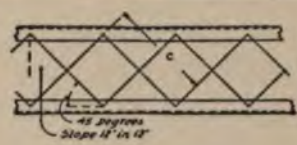
The sizes of lacing bars which have been proved to be efficient, depend upon the angles they make with the axis of the member, and upon their length. Table XV gives the sizes of such lacing bars:

TABLE XV

Size of Lacing Bars

SINGLE LACING

LENGTH C	THICKNESS OF BAR	WIDTH OF BAR
0 to 10 in.	$\frac{1}{4}$ in.	$2\frac{1}{2}$ in.
10 to 12 in.	$\frac{5}{16}$ in.	$2\frac{1}{2}$ in.
12 to 15 in.	$\frac{3}{8}$ in.	$2\frac{1}{2}$ in.
15 to 17 in.	$\frac{7}{16}$ in.	$2\frac{3}{4}$ in.
17 to 20 in.	$\frac{1}{2}$ in.	$2\frac{3}{4}$ in.
20 to 22 in.	$\frac{9}{16}$ in.	3 in.
22 to 25 in.	$\frac{5}{8}$ in.	4 in.
Over 25 in., use double lacing		



DOUBLE LACING

LENGTH C	THICKNESS OF BAR	WIDTH OF BAR
0 to 15 in.	$\frac{1}{4}$ in.	$2\frac{1}{2}$ in.
15 to 19 in.	$\frac{5}{16}$ in.	$2\frac{1}{2}$ in.
19 to 22 in.	$\frac{3}{8}$ in.	3 in.
22 to 26 in.	$\frac{7}{16}$ in.	3 in.
26 to 30 in.	$\frac{1}{2}$ in.	3 in.
30 to 34 in.	$\frac{9}{16}$ in.	4 in.
34 to 38 in.	$\frac{5}{8}$ in.	4 in.
Over 38 in., use $\frac{3}{4}$-in. angles		

Some specifications give the widths to be used. In case they do not, the above may be taken.

COLUMNS

46. **Strength of Columns.** The strength of a column depends not only on its **length** and on the **area of its cross-section,** but also on the **shape** and **position** of the material in the column. The writer took a sheet of the paper on which this book is printed, and, holding it out straight, pressed down with it upon a scale. The scale registered 1/4 pound when the paper crumpled. Another sheet was taken, and rolled into a tube

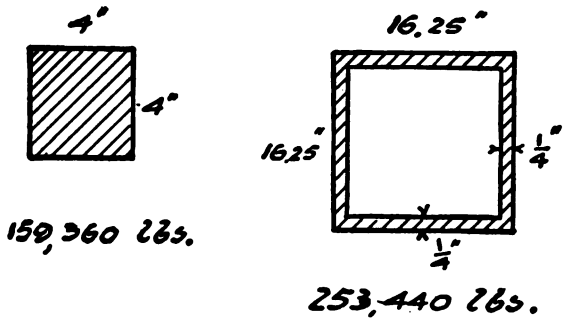


Fig. 66. Showing Distribution of Areas in Compression Members.

3/8 inch in diameter. The scale registered 26 1/2 pounds before the paper crumpled. Again, suppose that a column 12 feet long, consisting of a solid bar 4 by 4 inches=16 square inches in area, were used. The strength would be 159,360 pounds if designed according to the formula

17,100—57 l/r given in (C 141). Now, if the same amount of material were put into a column with cross-section 16¼ by 16¼ inches, and the sides were ¼ inch thick, the column would hold 253,-440 pounds, an increase in strength of 94,080 pounds, or nearly two-thirds of the original load. From this it will be seen that the farther away from the center of the column the material is located, the stronger is the column. Fig. 66 shows the two sections.

Radius of Gyration. In picking out a column, it is important, for any given shape, to know how the area lies in regard to the distance from its center. This is given for different shapes, and is indicated by the **radius of gyration.**

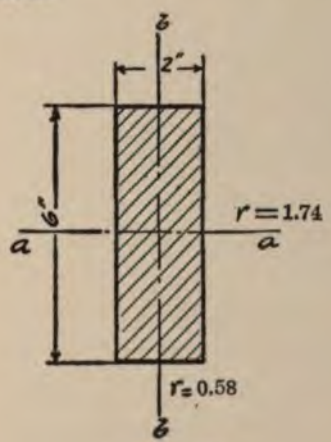


Fig. 67.

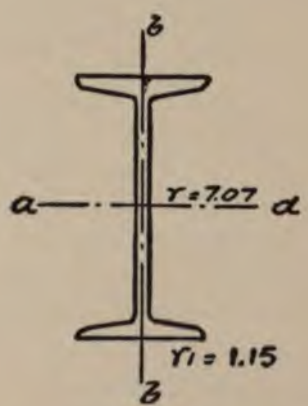


Fig. 68.

Illustrating Radii of Gyration.

It is important that the column be **symmetrical in section.** Any departure from this makes

the column weaker in one direction than the other. If a board 6 inches wide and 2 inches thick has a load put upon it, it will break on the broad side—that is, by bending about an axis parallel to the broad side. If the radius of gyration were computed for the axis **a-a** (Fig. 67) it would be found equal to 1.74. If it were computed for the axis **b-b**, it would be found equal to 0.58. This shows that the material is placed farther from the axis **a-a** than from the axis **b-b**. It also shows that the section is many times stronger about the axis **a-a** than about the other; and if it were supported on the sides so that it could not bend except about the

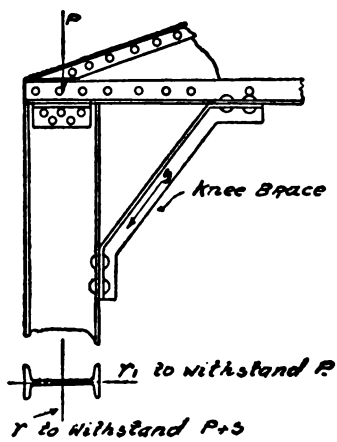


Fig. 69. Radius of Gyration in Column with Knee-Brace.

axis **a-a**, the column would hold a load several times what it would if it were left free to bend about the axis **b-b**.

For the reason that they are unsymmetrical,

single shapes such as channels, I-beams, etc., make very inefficient columns, as they are much weaker in one direction than in the other. Take an 18-inch 55-pound I-beam, for example. In (C 97-98), columns 9 and 10, are found the radii of gyration **r** and **r'** given as 7.07 and 1.15. This shows (Fig. 68) that the I-beam is much stronger about the axis **a-a** than about the axis **b-b**. Also, a comparison of I-beams of any given depth will show that the lighter I-beams are the most economical, since they have a greater radius of gyration than the heavier ones, thus showing that more strength per pound of metal is obtained from the lighter than from the heavier. For example, the 15-inch 42-pound I-beam gives **r**=5.95, or 5.95÷42=0.142 for each pound; while the 15-inch 100-pound gives **r**=5.53, or 5.53÷100=0.055 for each pound, showing that the lighter beam is about 0.142÷0.055=2½ times more economical.

An investigation of columns 9 and 10 (C 101-102) will show that the single channels are even more uneconomical than single I-beams. Columns like Fig. 65, **j**, are open to the same objection as single shapes. If, however, a column has a brace or some other member exerting a force against one side of it, and the smallest radius of gyration **r'** is sufficient for the vertical load, then column sections of single shapes are of use, since a larger radius of gyration **r** will be required to make the columns safe against the action of the thrust. The case of the column of

a mill building where a knee-brace is used, is an example of this kind of loading (see Fig. 69).

For columns carrying only vertical loads, some sort of section as is shown in Fig. 65 (**b** to **i**) is used. In case lacing bars or tie-plates are used, they are not assumed to take any of the load which comes upon the column. They are merely to hold the two sides of the column together, so that they will act as one shape. In cases where channels or I-beams are used with lacing bars or tie-plates, the radius of gyration **r** for the two shapes is the same as for one; but the radius of gyration about the axis **b-b** is different from **r'**, and it varies according to the distance apart the shapes are spread.

In order that the r_{b-b} may be equal to r_{a-a} and therefore have the column equally strong in both directions, the I-beams or channels must be spaced as given in column 14 of (C 98, 100, and 102).

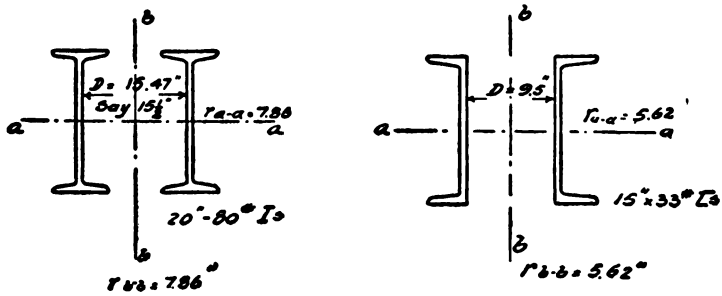


Fig. 70. Spacing for Equal Radii of Gyration.

To illustrate the above, take a column of two 20-inch 80-pound I-beams, and another of two 15-inch 33-pound channels (Fig. 70). The radius

of gyration about **a-a** is the same as for the one shape, and is found to be 7.86 for the I-beams, in column 9 (C 97); and 5.62 for the channels, in column 9 (C 101). In column 14 of their respective tables, the distance 15.47 inches and 9.50 inches are found. These are the distances which make $\mathbf{r}_{b\text{-}b}$=7.86 for the I-beam and 5.62 for the channel. If these distances were lessened, the $\mathbf{r}_{b\text{-}b}$ would be less than the values given; if the distances were made greater, the $\mathbf{r}_{b\text{-}b}$ would be greater in each case. The $\mathbf{r}_{a\text{-}a}$ will remain the same, no matter what the distance **D** is. In order to have an economical column, the distance **D** must never be less than that given in the Carnegie Handbook.

For any distance apart of the shapes, or for sections such as Fig. 65 (**d** to **j**), the radius of gyration for any axis may be determined by the following method:

(a) To the moment of inertia of each shape for an axis parallel to the axis to which it is desired to find the radius of gyration, add the value obtained by twice multiplying the area of that shape by the distance between the two axes mentioned above.

(b) Divide the result by the total area of the section.

(c) In the second column of (C 281-282) find the value which is nearest to the result obtained in **b**, but which has two more whole figures to the left of the decimal point. Look opposite this, and in the first column on the left-hand side of the page will be found a value. Point off one decimal place at the right, and the result is the radius of gyration.

Moment of Inertia. The moment of inertia

is a certain value for any given shape, and, somewhat like the radius of gyration, indicates the strength-resisting qualities of the section.

Let it be required to compute the radius of gyration for the section in Fig. 71. In column 6 (C 111), the distance from the center of

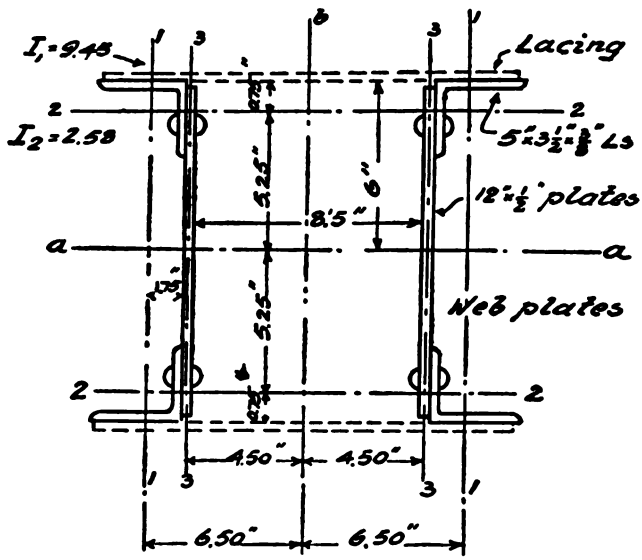


Fig. 71. Data for Computing Moment of Inertia.

gravity to the back of the long leg of the angle is found to be 0.75 inch for a 5 by 3 by ½-inch angle; and in column 7, the distance from the back shorter leg is 1.75 inches. The moment of inertia with respect to the axis 1-1 for one angle, column 9, is 9.45; while that for axis 2-2, column 8, is 2.58. The center of gravity of the web-plates is on the axis a-a.

The moment of inertia of plates is taken from

Courtesy of R. E. Gaut, Bridge Engineer, I. C. R. R.

GANTRY TRAVELER USED IN ERECTING A 375-FOOT SPAN OVER OSAGE RIVER.

On line of Chicago, Rock Island & Pacific Railroad between St. Louis and Kansas City.

Courtesy of R. E. Gaut, Bridge Engineer, I. C. R. R.

END POST BEING HELD IN PLACE WHILE PIN IS DRIVEN AT JOINT U_1.

Erection of a 375-foot span over Osage river between St. Louis and Kansas City, on line of St. Louis, Kansas City & Colorado Railroad, now part of the Chicago, Rock Island & Pacific Railroad System.

Plate 29—Steel Construction.

table (C 150). In the ½-inch column on the 12-inch line, is found 72.00 as the moment of inertia of one plate. The area of the section, Fig. 71, is:

2 Plates 12 in. by ½ in.	=12.00 square inches
4 Angles 5 in. by 3 in. by ½ in. (C 111)	=15.00 square inches
Total	27.00 square inches

Let it be required to obtain the radius of gyration r_{a-a}. Operation **a** is:

Moment of inertia of 4 angles = 4×2.58 =	10.32
Moment of inertia of 2 plates = 2×72.00 =	144.00
Angles multiplied twice by distance:	
4×3.75×5.25×5.25 =	413.39
Plate multiplied twice by distance:	
2×6.00×0×0 (since axis coincides with a-a)=	0.0
	567.71

Operation **b** is:

567.71÷27=21.03

Operation **c** is: On (C 281) near bottom, and in second column, is found 2116. In first column is 46. Therefore 4.6 is the radius of gyration.

In regard to the r_{b-b} it should be equal to or greater than r_{a-a}. Just to what distance back will be necessary to give this, is not known, and can be determined only by trial. A rough guess is to multiply the width of the plate by 0.7, and make it somewhere about that figure. In this case 12×0.7=8.4 inches—say make it 8½ inches. The computation will not be given, but this distance makes r_{b-b}=4.8, which shows the column to be safer about axis **b-b**. The distance might

be cut down to 7¾ inches, and a recomputation made. The result would probably be very close to 4.6. Usually two or three trials are necessary before it can be determined correctly. If the last computation comes within 1/10th of the other radius of gyration, and is greater, let it stand. Do not recompute.

47. **Design of Compression Members.** The strength of compression members also depends upon the length of the column, as is seen from the formula:

$$P=17.100-57\times\frac{\text{Length in inches}}{\text{Radius of gyration}}.$$

Here **P** is the given load which is allowable on each square inch of column area. This is the formula used in (C 125) for fixed loads and for buildings. For live loads and for bridges, use 6/10ths of the above.

As an illustration of the use of this formula, suppose the column of Fig. 71 to be 30 feet long. Then,

$$\begin{aligned} P&=17{,}100-(57\times30\times12)\div4.6\\ &=17{,}100-4{,}461\\ &=12{,}639 \text{ pounds per square inch.}\end{aligned}$$

The area of the section was 27.00 square inches, and therefore the column could hold $12{,}639\times27$ $=341{,}253$ pounds.

The design of compression members of Z-bar columns and channel columns, is conveniently made from tables (C 127-142). The information on (C 125-126) should be carefully read before

using them. These types are in very general use. In case other types are desired, a section must be assumed, and the radius of gyration worked out as shown above. The stress as given by the formula must then be divided into the total load on the column (or stress in the compressive member); and this result—which is the required area—must be compared with the actual area of the assumed section. If the assumed section is less than the required, or differs greatly from it, another section must be assumed and the operation repeated. This operation must be continued until the assumed and the required sections agree closely.

The student of Steel Construction will find it of great advantage if, in addition to the treatment here given, he will familiarize himself with the principles of the "Strength of Materials" as specially presented in any of the standard textbooks on that subject.

As an example, let it be required to design a **Z-bar column** to carry the load of 341,253 pounds which is the strength of the 30-foot column of Fig. 71. This is equal to 341,253÷2,000 =170.67 tons. On (C 129), in the table of 10-inch Z-bars, on the 30-foot line, 172.2 is found, and this is the nearest value which can be found and still not be less than the 170.67. Looking up at the top of the column, it is seen that the Z-bars and plate must be $^{11}/_{16}$ inch thick. Also it is found that the area of the section is 32.7 square inches. By looking on (C 103, 130, and

248), the sizes of the Z-bars and the plates are found, and the section is determined to be:

4 Z-bars 5 in. by 3¼ in. by $^{11}/_{16}$ in.=27.84 square inches
1 Plate 7 in. by $^{11}/_{16}$ in. = 4.81 square inches
32.65 square inches

In the table of 12-inch Z-bar columns on the same page of the handbook, is found 180.2 on the 30-foot length; and at the top of the column, the metal is found to be $^{9}/_{16}$ inch, and the area 31.2 square inches.

Both of these sections are greater than the section as designed in Fig. 71, which had only 27.00 square inches, but would be used if a Z-bar column is desired.

If a **channel column** is desired, (C 140) should be consulted. In the 8th column, and near the bottom of the page, 173.2 is found on the 30-foot line. This requires (see top of page to left of figure for width of plate):

(C 101) Two 9-inch 25-pound channels=14.70 square in.
(C 250) Two 11-inch $^{13}/_{16}$-inch plates =17.88 square in.
32.58 square in.

By (C 141), it will be seen that two 10-inch 20-pound channels, and two 12 by $^{13}/_{16}$-inch plates, give 177.9 as the capacity; and computations will show an area of 30.86 square inches. A smaller section yet is that from two 12-inch 20.5-pound channels and two 14 by ⅝-inch plates. The area for this section is 29.61 square inches. A length of 32 feet is here used at the top line

for values under that. The results of the areas are:

Z-bar	32.65 sq. in.
Channel column	29.61 sq. in.
Angles and Plates column	27.00 sq. in.

from which it would appear that the Z-bar column is the most expensive section, and the angles and plates the least. This, however, is not the case, for the small amount of riveting done in the Z-bar column—only two rows—more than offsets the saving in metal.

The channel section is next, with four rows of rivets; and the angles and plates are last, with four rows to connect the angles with the plates, and four rows to connect each side by means of lattices.

In some cases, bridge sections cannot have plates on both sides, as in the case of columns, since other members of the bridge must go inside of them. In such cases the plate is put on one side only (see Fig. 72). This changes the

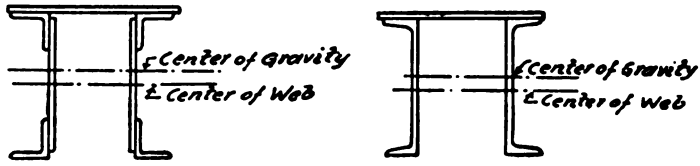


Fig. 72. Showing Shifting of Center of Gravity.

center of gravity of the section from the middle web to a position higher up; and the radius of gyration must then be computed as referred to this axis through the center of gravity. A great many sections of this character are given in

Godfrey's Handbook (see pages 104 to 119), and to this the reader is referred in case it is desired to design such sections.

48. **Rivets in Compression Members.** The spacing of rivets in compression members is not determined theoretically. The number of rivets must be such that sufficient should be put in between the center of the member and the end to transfer the stress from the angles to the webs or from the plates to the channels. In the tie-plates, they are usually spaced 3 inches apart, or more, a sufficient number being put in to fill out the plate.

In Fig. 71, one angle is 3.75 square inches in area. On page 243 the allowable unit-stress is computed to be 12,639. Therefore each angle carries $3.75 \times 12{,}639 = 47{,}396$ pounds. If the allowable unit-stress in shear were given as 7,500 pounds, one $\frac{7}{8}$-inch rivet would stand (C 195) 4,510 pounds, and $47{,}396 \div 4{,}510 = 11$ rivets would be required in the 15 feet on each side of the center. Here, as in most cases, the computed spacing would be too great, and the rivets are usually put in at a 4- or 5-inch spacing up to about three feet from the ends, where a 3-inch spacing is used for the remainder of the distance.

49. **Loading of Compression Members.** The loading of compression members should be central if the stress is brought on by means of a pin, as in a bridge the pin should be at the center of gravity of the member. If, as in case of columns, the load comes on top or from beams

which are connected to it, the load should be central and the connecting beams should be symmetrically joined in. Fig. 73 shows how connections should be made in the case of pin ends

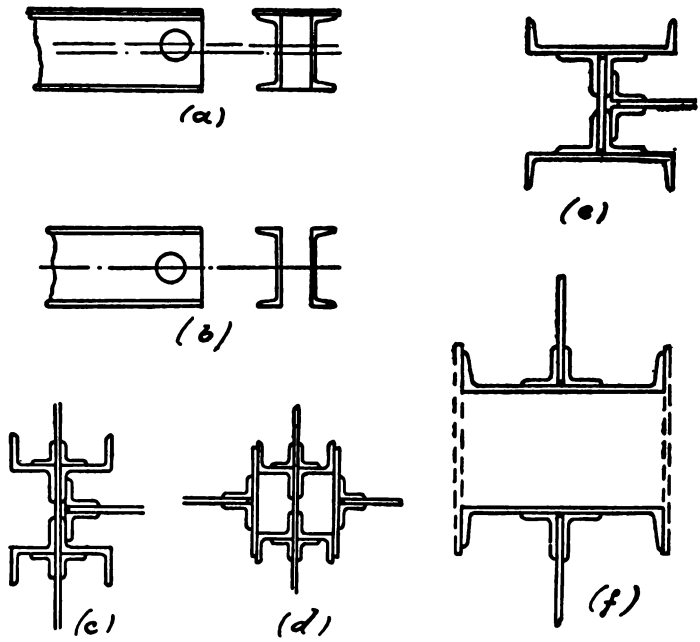


Fig. 73. Connections to Columns.

and also in the case where 1, 2, 3, and 4 beams are connected to the column.

If columns do not have connections symmetrical and, as in case of one beam, at the center of the column, the stresses will be greatly in excess of those as computed, and the member may fail. Also, the columns should be so placed that the reactions of beams coming on opposite sides of it will be equal or as nearly so as possible. Fig.

74 shows some common framing practice which should be avoided.

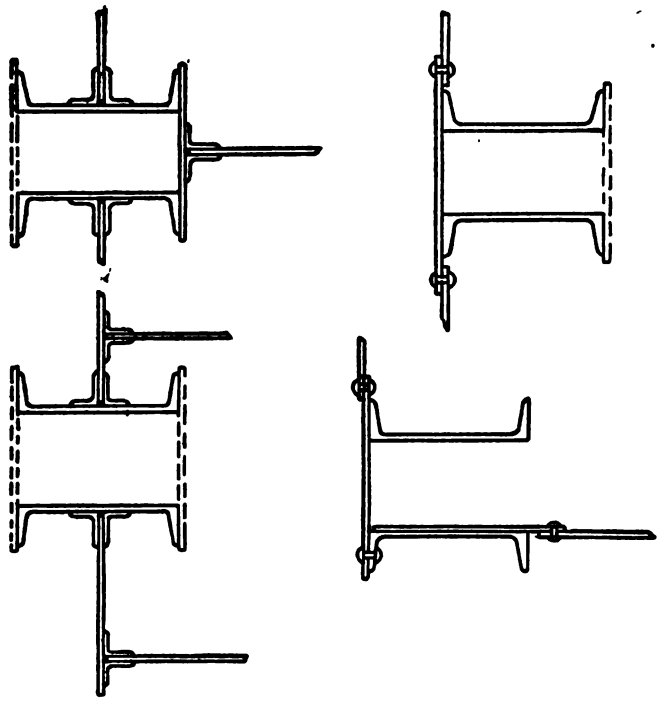


Fig. 74. Illustrating Incorrect Methods of Connecting to Columns.

CLASSES OF TENSION MEMBERS

50. Tension members may be divided into two classes—**simple members** and **built-up members.** The first class may be still further divided into **eye-bars** and **single shapes.**

Eye-bars are bars with circular ends, in which are holes whereby they are connected to other members by short circular bars called **pins.** The general form of the head, and also

the thinnest bar for any particular width, together with the largest pin to be used with a bar of a given width, are to be found in (C 212).

In order to make the head so that when the eye-bar breaks, it will break in the bar part and not in the head, it has been found necessary to make the sum of the distances **a** greater than the distance **w**, the width of the bar.

Several different proportions have been used in the past, but common practice now appears to have $2a=1\frac{1}{3}w$. That is, the area of the bar through the pin is $\frac{1}{3}$ greater than that through the body of the bar. If **P** is the diameter of any pin, then **H**, the diameter of the head for a width **w** of bar, is:

$$H=P+1\tfrac{1}{3}\,w,$$

and the result is taken to the nearest half-inch. See Fig. 75.

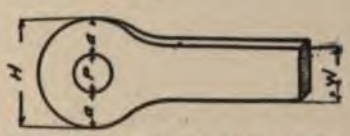


Fig. 75. Proportions of Eye-Bar Head.

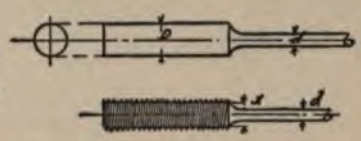


Fig. 76. Proportions of Upset in Bar.

As an example, let the width of a bar be 8 inches, and the pin be $6\frac{1}{2}$ inches. The diameter of the head is:

$$H=6\tfrac{1}{2}+1\tfrac{1}{3}\times 8=17.14, \text{ say } 17 \text{ inches.}$$

The pinhole should not be less than two-thirds the width of the bar, and the thickness of the bar should not be less than one-seventh the width of the bar. Bars which do not conform to

the above requirements are liable to be deficient in strength. The holes in the bar are usually made from $^1/_{50}$ to $^3/_{10}$ larger in diameter than the pin which is to go through them.

Eye-bars are usually in one piece, being the straight bar with the two heads. These heads may be made of different sizes. In some cases it is necessary to tighten up the eye-bars after they are in position; and then they are made in two pieces with screw ends (C 212), and these ends are connected by either a **sleeve-nut** or a **turnbuckle** (C 211). The former is preferable in light work, since, in the latter, ill-disposed persons—especially in the case of bridges—can loosen them up by means of a stick.

In case the stress is not great, round or square bars may be used. These usually have screw-threads cut on their ends, after first being **upset** at the ends to such a degree that after the screw-threads are cut the diameter **x** of the bar

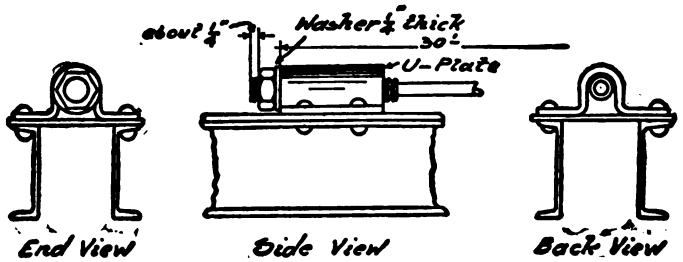


Fig. 77. Connections for Bars.

at the bottom of the threads will be slightly greater than the diameter of the bar **d** (see Fig. 76). This upsetting is done by first heating the

end of the bar, which is longer than it is required to be when finished, and then placing it in an upsetting machine, which forces the end in the form shown. In (C 205-206) is information regarding the upsetting of bars. The reader should carefully read the notes at the bottom of (C 206).

The connection of such bars to other members may be made by means of **clevises** (C 210), or by some form of **U-plate** as shown in Fig. 77. Sometimes a bolt-head is forged on one end of the bar, and a nut is used on the other end.

The weight of the bar itself may be computed

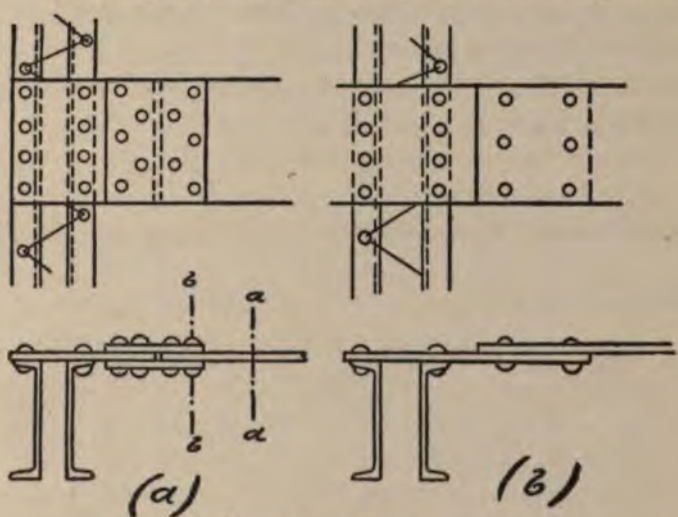


Fig. 78. Flat Bar or Plate Connections.

by help of the tables (C 261-266); and the weight of the head and nuts, from the table at the bottom (C 202). According to (C 206), last line on the page, it requires 6 inches additional on the

bar, to form an upset for a screw. The sizes of nuts are also given on (C 203-204). Of course the total length of the bar for which the weight is to be computed, must—in addition to the length required for distance from connection to connection, as in Fig. 77, where this is 30 feet—have the length required for the upset and the thickness of the nut, and then a small amount (say ¼ inch) additional to one or both ends, depending on whether there is a nut at one or both ends.

Another form of simple tension member is the **flat plate**, which is connected to other members at its ends by being riveted. It is coming more and more into favor; but, if used, the connection should be made as shown in Fig. 78, **a**, and not as in Fig. 78, **b**.

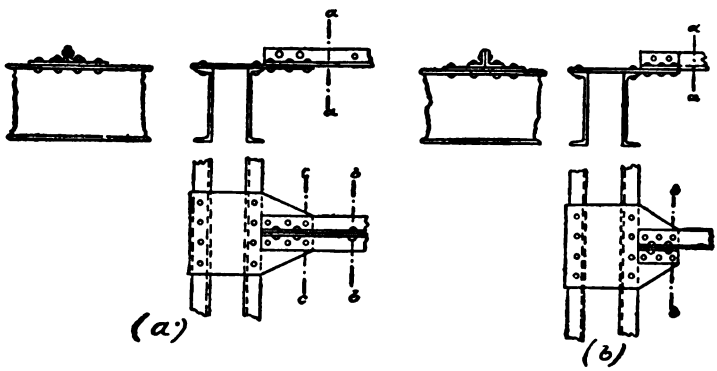


Fig. 79. Methods of Making Angle Connections.

Angles are much used as tension members, being riveted at their ends to the other members in a manner somewhat similar to the flat plate.

They should be used in pairs (Fig 79, **a**), or they should be riveted to both legs by means of a **clip angle** (Fig. 79, **b**). If they are riveted to one leg only, then, in the design, the other leg cannot be counted in as taking any of the stress, since the manner in which they are riveted causes exceedingly large stresses in the angle. It is customary in many cases to rivet the angle to one leg only. This should never be done, for only the large factor of safety prevents rupture of the member when it is stressed to the amount for which it is designed.

In all cases of tension members, the area which must withstand the stress is not at **a-a**, but at **b-b** or **c-c**, because the holes through which the rivets go cut out parts of the member and thus reduce its strength by the amount cut out.

The built-up tension members may be of any of the combinations of shapes shown in Fig. 65. The same remarks made under "Compression Members" regarding the rivets connecting the different shapes, apply here. At the ends, those classes of members may be connected either **by rivets** or **by pins** to other members. In case of their being riveted, care must be taken to see that there is enough area after all the rivet-holes in any cross-section are taken out.

In case of a pin connection (Fig. 80), care must be taken to see that the section through the pinhole is as great as that required by the design. Also, the distance from the center of the pinhole to the end of the member must be

carefully designed, or the member will fail by splitting out along the line **b-b**. In order to save material, it is advisable to have the section **a-a** through the center of the pin cut through no rivets, as is shown in Fig. 80. This section, as in

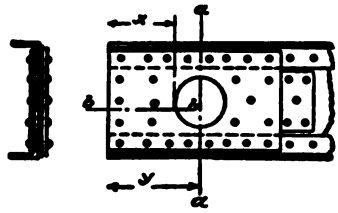


Fig. 80. Arrangement of Pin-Plates and End of Tension Member.

the case of the eye-bar, is greater than the section through the center of the pin, or the member would fail at that point. This amount of excess, as well as the distance **y**, is usually determined by the specifications, a very common requirement being that the section through the pin shall be one-fourth more than that through the center of the bar, and the section through **b-b** shall be at least equal to that in the body of the bar.

In tension members when compared with compression members, the reverse is true as regards the radius of gyration. Although attention is seldom paid to the fact, nevertheless the smaller the radius of gyration, the better the section. However, the radius of gyration is never computed for tension members.

If the web were thin, the pin would tear out of it somewhat after the fashion of a huge rivet. In order to prevent this, one or more plates,

called **pin-plates,** are put on as shown in Fig. 80, to prevent this. These pin-plates should be on compression members. The design of their thickness is the same for both classes of members, and the determination of the required number of rivets is the same; but in tension members, only sufficient rivets should be to the right of the section **a-a,** so that their combined strength will be equal to the section of the pin-plates through the section **a-a.** The remainder of the required number should be to the left of section **a-a.** The reverse is true for the distribution of rivets in compression members. This is due to the fact that the direction of the stress is just opposite in one case to what it is in the other.

51. **Design of Tension Members.** In all cases the required area is equal to the total stress divided by the allowable unit-stress. Examples are given below, showing the method of design of different classes of members.

(1) A tension member has a total stress of 449,500 pounds. The allowable unit-stress is 16,000 pounds per square inch. What eye-bars would be used, and what would be the diameter of their heads provided a 5¼-inch pin was used?

Required net area=449,500÷16,000=28.10 square inches. Let 4 bars be used. This number will be indicated by the condition of design and class of structure, but is usually an even number. Area of one bar=28.10÷4=7.03 square inches. On (C 247), in the 6-inch column,

is found 7.13 on the $1^3/_{16}$-inch line. It is to be noted that $1^3/_{16}$ is greater than $^1/_6$th of 6. Therefore 4 of these bars will be used.

Diameter of head, $H=5\frac{1}{4}+1\frac{1}{3}\times 6=13\frac{1}{2}$ inches, being to the nearest half-inch.

(2) The stress is 48,000 pounds. If the tension member is to be one plate, what is the size, the allowable unit-stress being 15,000 pounds per square inch?

Required net area $= 48{,}000 \div 15{,}000 = 3.2$ square inches. By (C 247) it is seen that a $6\frac{1}{2}$-inch by $\frac{1}{2}$-inch plate with an area of 3.25 square inches, is the nearest. This is the net area. Since the plate is attached by rivets, there must be an increase in width equal to the diameter of the rivet used times the number which could be cut by a line drawn across the member. Suppose two $\frac{7}{8}$-inch rivets were cut. The width would then be $6\frac{1}{2}+2\times\frac{7}{8}=8\frac{1}{4}$ inches. An $8\frac{1}{2}$-inch plate would be used. Note that in this case the plate may be thinner than one-sixth the width. It is because no heads are forged on it as in the case of eye-bars.

(3) If the total stress and unit-stress are as above, design an angle to take the stress, the rivets being $\frac{7}{8}$-inch in diameter.

The required area will be the same, 3.2 square inches. An angle (C 113) $3\frac{1}{2}$ by 3 by $^{11}/_{16}$-inch will be tried.

Gross area = 4.00 square inches
Out for one rivet $1\times{}^{11}/_{16}$ in. = 0.69 square inch
Net area = 3.31 square inches

ERECTING A CANTILEVER SPAN.

View Showing position of derrick car during erection.

Courtesy of Chicago, Milwaukee & St. Paul Railway.

CANTILEVER SPAN NEARING COMPLETION.

The $^{11}/_{16}$ inch is multiplied by 1 (inch) instead of $\frac{7}{8}$ (inch), since, on account of the injurious action of the punch on the metal, the metal directly around the hole is not considered of value in withstanding stress, and so a portion $\frac{1}{8}$ inch larger than the diameter of the given rivet is taken out.

The angle chosen is the one that will be used. Other angles may be chosen and investigated; but it will be found that they will not give areas as near to the required, and at the same time greater. A clip angle of the same size will be used to help connect it at the ends.

(4) If the total stress is 240,000 pounds, the allowable unit-stress 16,000 pounds per square inch, and the rivets $\frac{7}{8}$-inch, design a section consisting of plates and angles. The pin at the end is 6 inches in diameter.

Required net area=240,000÷16,000=15.00 square inches. A common proportion is to make the plates somewhere near one-half this value. Two 12 by $\frac{3}{8}$-inch plates give a gross area of (C 250) 4.50×2=9.50 square inches.

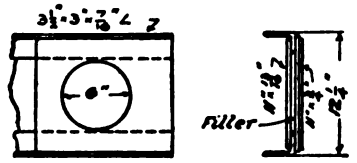


Fig. 81. End of Tension Member.

A sketch of the section is shown in Fig. 81. The pin-plate will probably require two rivets to go through the web on each side, making 8 in

all to be taken out of the section. The net area of the plate is:

$9.5-(8\times\frac{3}{8}\times1)=6.5$ square inches.

This leaves $15.00\div6.5=8.50$ square inches to be divided among 4 angles; and accordingly each angle must be $8.50\div4=2.13$ square inches in net section.

By consulting (C 113), a $3\frac{1}{2}$ by 3 by $\frac{7}{16}$-inch angle appears to be about correct. It will be tried.

Gross area, 4 angles $=4\times2.65=10.60$ square inches
Out for rivets, $4\times\frac{7}{16}\times1=$ 1.76 square inches
Total net area $=\overline{15.34}$ square inches;

or, $8.84\div4=2.21$ square inches each, which is as near as can be obtained. They will be used.

The total net area is:

Plates	6.50 square inches
Angles	8.84 square inches
Total	$\overline{15.34}$ square inches

At the section through the pinhole, the pin takes out $6\times\frac{3}{8}\times2=4.5$ square inches; and so the actual section there is only $15.34-4.50=10.84$ square inches, when it should be $15.00\times1\frac{1}{4}$ (see page 254) $=18.75$ square inches. This shows that plates must be added to make up the difference of $18.75-10.84=7.91$ square inches.

As has been said, the pin is as a huge rivet. The bearing value follows the same rule as the rivet (see page 134). If the allowable unit-stress were 15,000 pounds per square inch, the bearing value of the pin in the $\frac{3}{8}$-inch web would

be $15{,}000 \times 6 \times \frac{3}{8} = 33{,}750$. Since there are two sides or webs, the value of both would be $2 \times 33{,}750 = 67{,}500$ pounds. This shows that there is a difference of 240,000-67,500=172,500 pounds to be taken up by pin-plates. For one side of the member there would be $172{,}500 \div 2 = 86{,}250$ pounds. This requires $86{,}250 \div 15{,}000 = 5.75$ square inches bearing on the pin. Since the pin is 6 inches in diameter, the bearing area must be $5.75 \div 6 = 0.96$ inch thick. This is practically 1 inch. This will be made up of two 11-inch by ½-inch plates placed as shown in Fig. 81.

The small plate in between the web and the outer plate, is used only to fill in, since it is only as great in width as the diameter of the pin.

The net area of the other two plates is:

2 Plates, $2 \times 11 \times \frac{1}{2}$ in.....	11.00 square inches
Out for pin, $2 \times \frac{1}{2} \times 6$.....	6.00 square inches
Total	5.00 square inches

This is less than the 7.91 extra square inches required on the condition that the section through the pinhole shall be 1¼ times that required for the body of the bar. The plates will have to be made thicker. Each $^1/_{16}$ added will add $(11-6) \times {}^1/_{16} = {}^5/_{16}$ square inch to the net area; and since 7.91—5.00=2.91 square inches extra is required, it will be necessary to increase the thickness $2.91 \div {}^5/_{16} = 9$ sixteenths, making the total thickness of both plates $1{}^9/_{16}$

inches. The inner plate will be made $\frac{3}{4}$ inch, and the outer one $\frac{13}{16}$ inch.

The distance from the pin to the end shall be such that the area will be 15.00 square inches, the same as in the body of the member. The thickness is that of a $\frac{3}{8}$-inch web and the two pin-plates. This is $\frac{3}{8}+\frac{13}{16}+\frac{3}{4}=\frac{31}{16}$, and the distance must then be $(15.00+\frac{31}{16})\div 2=3\frac{7}{8}$ inches. The distance from the center of the pin to the end is then $6\div 2+3\frac{7}{8}=6\frac{7}{8}$ inches.

The web is $\frac{3}{8}$ or $\frac{6}{16}$ inch; the outer plate is $\frac{13}{16}$ inch; and the inner plate is $\frac{3}{4}$ or $\frac{12}{16}$ inch, making a total of 31 sixteenths. The stress distributes itself over the pin evenly, and each plate takes a stress in proportion to its thickness. The stresses in the pin-plates on one side of the member only are considered, those on the other side being the same.

$$\tfrac{13}{16}\text{-in. plate}=\frac{240{,}000}{2}\times\tfrac{13}{31}=50{,}300$$

$$\tfrac{3}{4}\text{-in. plate}=\frac{240{,}000}{2}\times\tfrac{12}{31}=46{,}450$$

As both of these plates get their stress from the web, there must be sufficient rivets in bearing in the web to transfer this. The required number (see second table, C 195), is:

$$50{,}300\div 4{,}920=11$$
$$46{,}450\div 4{,}920=10$$

Now each plate has a tendency to slide along the web. This will tend to shear the rivets off, and

therefore there will be required (see same table):

For $^{13}/_{16}$-in. plate, 50,300÷4,510=12
For $\frac{3}{4}$-in. plate, 46,450÷4,510=11

This makes a total of 21 for bearing, and 23 for shear. The distribution of the rivets is as shown, remembering that one rivet is in double shear but cannot take 2×4,510, but only 4,920, since it is in bearing in a $\frac{3}{8}$-inch web.

52. **Connections.** The connection of members by means of pins has been discussed. The connection of plate and angle members by means of rivets should by this time be easy of computation for the reader. The total stress is divided by either the shearing or bearing of a rivet, whichever is the smaller; and the result will be the number of rivets required.

Standard Connections. The connection of beams to girders and girders to columns has been discussed. The method given was a general one, and is applicable to all cases. However, for cases such as usually occur in building work, **standard connections** may be used. These are standard for any given beam, and are so designed that, if the beam will be safe for bending, the standard connection is safe. They are given on (C 177-188). They should not be made use of at random. First, the number of rivets required should be computed; and if this is the same as or less than those in the standard connection, then the standard connection should be used.

On (C 189-190) are given beam connections to Z-bar columns. They may also be used on channel columns or on columns built up of shapes. The range of values for reactions of beams is from 4.4 to 26.5 tons, and this is usually as great as is required. In case the reaction is greater than that there given, the number of rivets may be increased proportionately, or connection angles may be put on the beam as shown in Fig. 82, the additional rivets being put

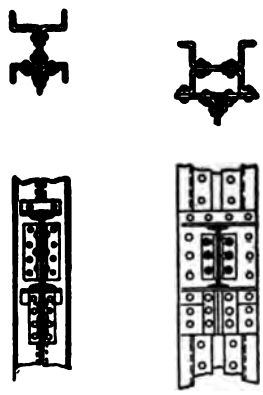


Fig. 82. Connections to Z-Bar Columns.

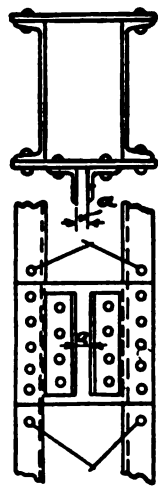


Fig. 83. Entering Connection.

in the connection angles. The field rivets are used with the top angle, in order that, should the beam be a little greater in depth than that given in the handbook, the adjustment can be made in the small angle and not in the beam or by cutting off the angle, which would be the way if field rivets were not used.

Entering connections, of which Fig. 83 is an example, should be avoided. The distance **a** is either made equal to the thickness of the web of the beam which is to join at that point, or it is made larger. In the first case, the web is more than likely to be thicker than the value given in the handbooks; and one of the angles must be cut off in order to get the beam in. On the other hand, if the opening is too great, there is the danger of a loose fit; but this can be done away with to a certain extent by putting in pieces of **shiming sheets.** Taking into consideration all conditions, it should be a fixed rule that they should never be employed.

CLASSES OF STRUCTURES

53. **Use of Beams.** The use of beams composed of channels and I-bars is practically limited to building construction. On account of their limited section modulus, their span cannot be great, since the moment becomes too great for their capacity. Also, in building work, the span is in many cases further limited by the plastering on its under side. Rolled beams are seldom used in railroad work for spans greater than 20 feet.

54. **Use of Girders.** The term **girder** here signifies the built-up member. Girders may be built up to spans of 150 feet; but beyond 140 feet, it is a question if their use is economical, since their depth is so great that web-plates cannot be rolled with lengths much greater than

the width, and this necessitates many splices. These splices are expensive. Again, such girders require large flange areas—usually greater than can be furnished by the thickest 8 by 8-inch angles—and plates and channels are then used. The webs of the channels are so thin that a great amount of riveting is necessary.

For spans up to 135 feet, girders are in almost universal use in buildings and in railroad construction. For highway purposes they have not come into very general use, but probably will do so in the near future.

BRIDGES

55. **Use of Bridge Trusses.** Since 1854 the metal bridge truss has been in general use for all spans from 10 or 15 feet up to several hundred feet. The huge Firth of Forth bridge in Scotland, and the Quebec bridge crossing the St. Lawrence river near Quebec, have spans of about 1,800 feet.

Bridges may be considered as **trussed beams.** A **truss** is a combination of members which take stresses in direction of their length only. Another definition, perhaps, is that a truss is a combination of tension and compression members. Two or more trusses constitute a **bridge.** The trusses constituting a bridge are connected by a system of beams on which the load is carried. This is the **floor system** of the bridge, and has no purpose except to carry the loads to the trusses.

56. **Classes of Bridges.** Bridges, like beams, may be divided into **continuous, simple,** and **overhanging.** In the case of overhanging bridges, there may be one or both ends overhanging. Such bridges are called **cantilever bridges** from the fact that the overhanging end is a cantilever (Plate 7). Other classes of bridges are the **suspension, swing, rolling-lift,** and **trunnion-lift.**

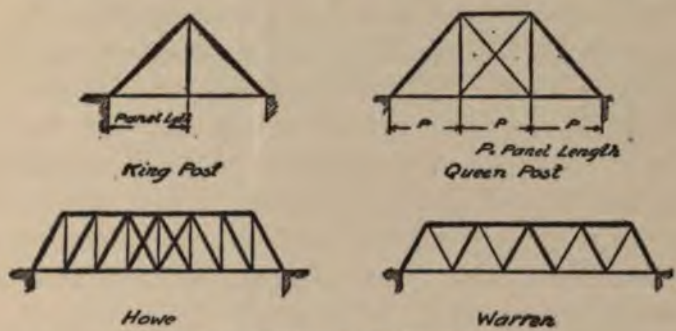


Fig. 84. Classes of Bridge Trusses.

Probably 95 per cent of all bridges built are of the simple type—that is, having each end resting on a support. The simplest forms are the **king-post** and **queen-post trusses** (Fig. 84). Since the panel length—that is, the distance between any two joints—is limited to about 20 feet, the greatest spans of these classes of bridges are 40 and 60 feet respectively.

Other classes of simple trusses are the **Howe, Warren, Pratt** (Plate 8), **Camel Back** (Plates 9 and 10), **Baltimore,** and **Petit** (Plate 11). The first three and the last were named after their inventors; the fourth, because of its

humped back; and the fifth, because the Baltimore Bridge Works was its first builder. The camel back is frequently called the **bowstring.**

The limiting economical span for a simple truss is between 300 to 500 feet, according to the class. The Pratt is built to a greater extent than any of the others, probably 85 per cent of all simple trusses are of the Pratt type. The Petit and Baltimore are for long spans only—say from 250 to 500 feet. The rolling-lift (Plates 12 and 13) and trunnion-lift (Plates 14 and 15) are the outcome of the attempt to do away with the old swing bridge or **drawbridge** (Plate 16). The latter was costly, and also had a pier in the center; and this, especially in narrow streams, was a great obstruction to navigation.

The rolling and trunnion lifts have their foundations and supports on the banks, the full width of the stream being available for navigation. These bridges are balanced about their rolling or trunnion centers, by huge counterweights of cast-iron blocks or masses of concrete, and are opened or closed by a pull or push on the operating strut, which is usually supplied with power from electric motors in a small house as indicated.

All bridges shown in Plates 9 to 16 are called **through bridges,** since the traffic moves through them, between the trusses. Any of them shown, except the suspension bridge (Plate 17), might be turned up the other way, allowing the

traffic to move over the top. They are then called **deck bridges.** Plate 18 shows a deck bridge of the Pratt type.

Continuous bridges have the same objections as continuous beams. A slight settlement of the trusses causes the stresses to be greater than computed. They are not used in this country, except in a very few cases.

Cantilevers and suspension bridges should be used for long spans only, or where their form offers ease of erection. They are very uneconomical for short spans. Their special use is for spans of about 500 to 1,800 feet for cantilevers, and 1,000 to 3,000 feet for suspension bridges.

In Fig. 84, the heavy lines indicate compression members, while the light lines indicate tension members. Dotted lines indicate that the member sometimes takes compression and sometimes tension. The names of the various members are also put upon them, in many cases. In simple trusses, the top chord is always in compression, and the bottom chord always in tension.

Design of Bridges

57. The determination of stresses in steel bridges is one of the functions of the designing engineer, and calls for a high grade of technical skill and mathematical training.

Once the stresses are computed, however, the design is simple. The design of the sections

is made in the same way that the tension and compression members are designed in this text. The economical design and the getting the parts to fit together in the best manner, can be done only after much experience. The student is advised to study plans of actual bridges and more advanced texts, before attempting design.

Both ends of a bridge should be bolted to the supports so that they cannot move. In warm weather a bridge is slightly longer than in cold weather, and any change of temperature causes it to change in length. If both ends were fixed, the stresses in the bridge would change considerably from those computed on the basis of the loads, and may in extreme cases cause danger. One end should be fixed to the support, and the other end should either be on **rollers** or on a planed **plate** so that it may move backwards and forwards when the temperature changes.

MILL BUILDINGS

58. Buildings which are used for manufacturing purposes are usually classed as **mill buildings**. They may be entirely of steel, or they may have brick walls and steel roofs. The steel roofs may be covered with a steel roof-covering or with slate, shingles, tile, concrete, or any other roof-covering.

In any case, mill buildings usually consist of a series of **bents** (Plates 19 and 20), which consist of a pair of columns supporting a **roof truss** and, in many cases, a **crane** also, which runs

lengthwise of the shop. In some cases the columns carry the wall covering; in other cases the columns carry the roof truss only, masonry walls being built in between them; while in still other cases, there are no columns at all, the roof trusses resting directly upon the tops of masonry walls.

The roof trusses are connected by beams on which the roof cover is supported. These are called **purlins.** The columns are connected by members running horizontally, and these members are the supports of the walls of the building. They are called **girts** (see Plates 19 and 20). The space between any two bents is called a **bay.**

The covering is usually of **corrugated iron.** Since the strength of the iron is such as safely to carry a man if it is supported at its end by supports 5 feet apart, the spacing of the purlins on the truss should not be greater than 5 feet. The girts must be strong enough to carry the weight of the siding, and also to withstand the **wind pressure.** This wind pressure is usually assumed at 30 pounds per square foot of wall surface. The spacing of girts should be limited to 5 feet in order not to have the corrugated steel stressed too greatly by the wind.

The length of bays is an important factor in the cost of the building. This varies with the span of the roof truss. For spans of from 10 to 20 feet, the bays should be 10 to 12 feet in length; for spans of from 30 to 60 feet, 15 feet

in length; for spans of from 60 to 80 feet, 20 feet in length; and for spans of from 80 to 150 feet, 21 to 25 feet. In many cases, wooden instead of steel purlins are used; and then the span is one foot short of the standard lumber lengths, being 11, 13, 15, 17, and 19 feet, in order that 12, 14, 16, 18, and 20-foot joists respectively may be used for purlins.

The **pitch** or **slope** of the roof truss is important. In case the roof-covering is of tar or pitch and gravel, it should be small, or on warm days the roof-covering would run. In case tile, shingles, or corrugated iron are used, the pitch should be steep, or the rain will beat up under the covering at its laps, and cause leaks. For concrete roofs, the slope may be small or great as desired. It might be said that for pitch or tar roofings, the pitch of the roof should be $^1/_5$ or less, while for the other classes it should be over $^1/_5$. A usual pitch for the latter class is $\frac{1}{4}$ or 30 degrees, either of which gives satisfactory results.

The effect of the pitch of the truss on the weight is small for slopes from $\frac{1}{6}$ to $\frac{1}{3}$, provided the lower chord of the truss is horizontal. If the lower chord is raised, the difference will be considerable, and comparative design should be made to determine the most economical pitch.

Purlins are most economical when made of Z-bars, in which case the section modulus given in Table X should be used.

Channels and Z-bars are the best for girts,

or, if the length of bays be great, I-bars may be used. Angles have been used in some cases, but they are not economical.

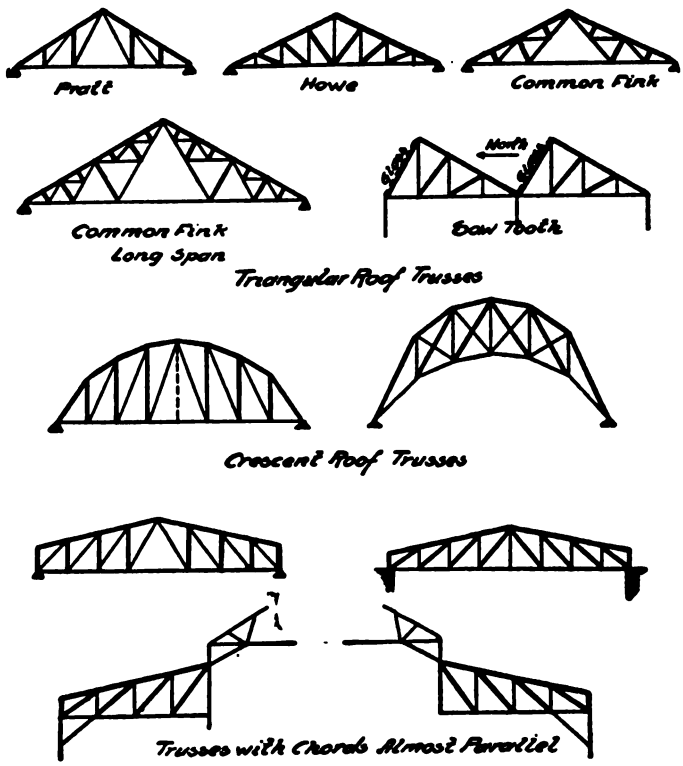


Fig. 85. Classes of Roof Trusses.

59. **Design of Roof Trusses.** There are several types of roof trusses (see Fig. 85). The design of these is made after the manner previously given, the stresses being once determined. The loads are given in the specifications, and the approximate weights are also given.

The stresses may be determined by the methods of mechanics; or, for the most usual forms and slopes, they may be determined by (C 173-174). The methods given are for trusses which rest upon masonry walls. In the case of stresses in trusses which rest upon columns, the stresses are greater in some members than in others. The computation of such trusses is rather complicated; but if the stresses in the members **bc** and **bf** as computed by (C 174) are doubled, and the stress in **gc** is increased by ½, the trusses will be safe, and will do for all practical purposes.

Most of the members of roof trusses of ordinary spans, consist of one or two angles. If one angle is used in either tension or compression, tables (C 109 to 119) may be used. If two angles are used in compression, the tables in (C 144-146) are to be used, the least value of the radii of gyration being employed. Usually the angles are ½ inch apart, owing to the fact that the connection plate goes between them. In such cases, use r_0 or r_2, whichever is the least.

Connections of members are made in most of the ordinary spans—and should be in all spans up to 125 feet—by means of rivets. In this case the bearing value of the rivet in the connection plate, or its value in single shear, is used, taking the smaller value if the case is one where one angle joins the plate. If the member consists of two angles, then the value of the rivet in bearing in the plate determines the number

Courtesy of Scherzer Rolling-Lift Bridge Co.

ERECTION OF ROLLING-LIFT BRIDGE IN OPEN POSITION.

PLATE 32—STEEL CONSTRUCTION.

Courtesy of R. E. Gaut, Bridge Engineer, I. C. R. R.

TWO VIEWS SHOWING USE OF CANTILEVER TRAVELER.

PLATE 33—STEEL CONSTRUCTION.

of rivets. In each case, whether the member is in compression or in tension, the number of rivets is obtained by dividing the total stress in the member by the value of the rivets.

Care should be taken to see that the gauge lines of angles, when three or more meet at a joint, should all meet at the same point, as in

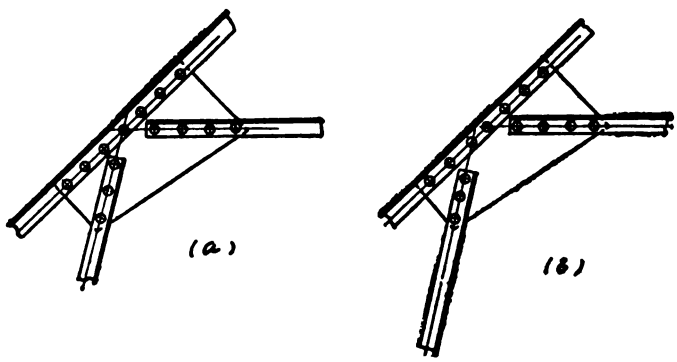


Fig. 86. Correct and Incorrect Methods of Connecting Up Three or More Angles that Meet at a Joint.

Fig. 86, **a**, and not as in Fig. 86, **b**. When they meet as in Fig. 86, **b**, extra stresses are set up in the members and the plate; and these stresses are additional to those caused by the loads on the truss. This is a quite common fault, and, in addition to showing poor design, may in extreme cases cause failure of the joint.

Clip angles should be used in all cases where an angle connects to a plate.

OFFICE BUILDINGS

60. An office building may be defined as any building which is of such height as to necessitate

a steel frame. Steel frames become a necessity when the height of the building becomes such as to require a masonry wall to be so thick as to take up valuable room which could otherwise be used for renting purposes.

The columns, and the joining and spacing of the floor-beams and girders, have already been discussed.

The great advantage of this type of building is that the walls may be made very thin—8 to 12 inches—since they extend only one story. This is made necessary by the fact that if they did extend more than one story, the expansion of the steel and the masonry would cause cracks in the walls, as these substances expand and contract at different rates under changes of temperature.

Another advantage is that the work of building the walls may begin at any story without in the least affecting the work at the other stories (see Plates 21 and 22). Still another advantage lies in the fact that since the steel is so strong, the columns may be placed far apart, thus leaving large, open spaces which may be divided up into rooms of any size after the building is completed; or, after they have been once divided, the old partitions may be taken out and a new division made to suit the new tenant.

The beams or combinations of beams which go between the columns at the outer walls, and which carry the outer walls, are called **spandrel beams.** The wall for the next story above is built into angles and various other shapes which

are attached to the spandrel beams. Plate 23 shows spandrel beams.

61. **Bents and Towers.** When the end of a structure rests upon a two-legged tower, the structure is said to be supported by a **bent.** A familiar example of this is found in the case of elevated railroad structures. The legs of this bent are columns, and carry the reactions of the structure and its loads.

If there is no wind, the above-mentioned loads are the greatest that come upon it; but if the wind blows sidewise on the structure, additional stresses occur. The stress in the leg on the side where the wind blows is decreased, while that in the opposite leg is increased by a like amount. If the bent is high, this effect may be such as to lift one side; and in such cases, the ends of the columns must be bolted down to a large block of masonry or fixed in iron blocks. The columns must also be designed to withstand this additional stress.

Likewise, in towers, great stresses occur from the wind; and the columns must be designed for the extra stresses brought upon them, and must also be fixed so as to prevent the tower blowing over.

62. **Steel Grillage Foundations.** In order that the wall or the end of a column may not sink into the soil, steel beams are laid in concrete as shown (C 165). If the pressure comes from a column, a masonry footing may be built on top of the beams, and the same method used.

On (C 165-166) are instructions, examples, and tables illustrating the design of such footings.

THE MAKING OF STRUCTURES

63. **Designing and Drafting.** When bids have been asked for by parties, the firms desiring to secure the work obtain a set of **specifications** from the parties desiring the work done, and also all data regarding the location, dimensions, and layout. In some cases, the parties desiring the work done furnish a drawing showing the general class of structure they desire, and, if the structure has many peculiarities, they may also furnish drawings giving the stresses in all of the members, and also the sizes of them. In case the drawing, with the stresses and sizes of members, is not furnished, the contractor makes one for himself, and then makes sketches from which he can very closely estimate all the material required. He then prepares his bid.

If his bid is accepted, he then sends this sheet, called the **stress sheet,** to the drafting room, where the detail drawings or shop drawings are to be made. As soon as a contract is secured, a certain number is given it. This number is put on all drawings and correspondence relating to it, and is painted on all material for the members of the structure. As soon as the stress sheet reaches the drafting room, one of the best men takes it, and from it orders all the material necessary to build it. This bill of material is sent at once to the rolling mills, and

the stress sheet is turned over to the draftsman who makes the shop drawings.

The **shop drawings** show the size, dimension, and position of each piece that goes in the structure, and also the position of each rivet and rivet-hole. The draftsman also makes out a list of all the rivets required for the field and shop, and also a shipping bill which gives the length and size of each member of the structure.

After the draftsman finishes his work, it goes to the checker, who carefully goes over it and corrects the mistakes, if any. The tracings are then sent to the blue-print room, and several sets of prints are made from them. One set is sent to the purchaser for his approval; and after it is returned approved, sets are sent to the heads of the various shop departments which will have anything to do with the structure. One set is kept on file in the office.

64. **Making the Members.** The **templet shop** begins work upon the order. Here holes are bored in pieces of boards so that when these boards are clamped to the piece of metal which is intended for part of a member, the holes will come where the holes should be in the member, and the ends where the end of the member should be. After the templet is made, it is marked with the order number and with the piece number, and has the notation signifying the kind of rivets and where the various cuts should be. It is then sent to the **laying-out shop** or **division.**

In many cases the steel for the structure arrives from the mills long before the drawings are made. It is unloaded, and placed in piles in the yard or in a shed near the laying-out shop. When the time comes that the templets are done and begin to come in, the material is brought in, one piece at a time as desired, and, if necessary, is straightened. The layers-out then take a piece, put it on their bench or trestle, clamp on the templet, and, with a center-punch and hammer, make a small indentation in the steel in the center of each rivet-hole by placing the center-punch in the hole bored in the templet and striking it a sharp blow with a hammer. The piece is also marked where it should be cut off, and also with notes regarding the rivets.

From here the piece goes to the **shears,** where it is cut to shape; and then to the **puncher,** where the holes are punched $^1/_{16}$ inch smaller than the diameter of the rivet which is to go in them.

From here they go to the **assembling department.** Here the different pieces are cleaned of rust and scale, and painted one coat of some good paint on the surfaces which are to be riveted together. The different pieces which go to make up a member are then bolted together by bolts placed in the rivet-holes at frequent intervals. The member is now ready for the **reamers.**

The reaming machines are usually placed close together in rows, so that several of them can work on the same member at the same time. These reamers are nothing more or less than

boring machines. They bore out the rivet-holes to a size $^1/_{16}$ inch larger than the diameter of the rivet. Reaming bores out that material which was the most injured by the punching, and it also brings into line any holes which do not exactly match up.

From the reamers, the member goes to the **riveters,** and is riveted together, the assembling bolts being taken out to make way for the rivets.

The whole member is next taken to the **rotary planers,** where the ends are cut to the exact length. Also pinholes, if any, are bored with the boring machines.

From here it is taken to the **paint shop,** where all rust and dirt are cleaned off, and it is given the coat of paint as required by the specifications.

The member is now ready to be shipped. It is loaded on the cars, care being taken to see that it will clear all objects along the track or in the tunnels, if any; and also to see that it is securely fastened for its journey. In case a structure is to be shipped by steamer, a diagram of the boat, showing all hatches, stanchions, and pillars in the hold, should be furnished before the drafting has begun, in order that the various members should be so built as to go in the hold.

65. **Inspection.** From the time the drawings are first sent out, all the processes of construction are under close double inspection—that of the company's inspector, and that of the

inspector employed by the purchaser. The inspectors first look over the drawings closely, and then see that the material, the methods of working, and the results are in accordance with the specifications. Having a higher education than many of the shop men, they understand the drawings better, and therefore should be present not only to criticise but to help along wherever possible by giving advice when asked and by making kindly suggestions which will make the work easier for the workmen.

COST OF STEEL STRUCTURES

In bidding on a structure, the bid may be for a lump sum to complete the work, or for so much per ton, or for so much per pound. In any case, the contractor will not directly estimate it that way himself. He will take into account and figure on the ten or twelve costs which go to make up the total, and will then transform them into the lump sum bid or the unit-price. These various operations, together with their costs, are given in the following articles.

66. **Raw Material and Mill Work.** The raw material means the various shapes just as they come from rolls. These vary in price in different years, and at different periods of the year. They also vary in price according to the size and kind of shape. Since the variation in price is due chiefly to the variation in the price of ore and cast iron or billets, there is a certain price for various shapes called the **base price;**

and to this must be added the so-called **card extras** for a particular shape, in order to get the price per pound for that particular shape.

The base price will be found in standard trade papers covering the steel structural field. The card extras are issued at certain intervals by the manufacturers, and can be had for the asking. Any dealer in hardware can get you one. Sometimes one-half the card extras are to be added to the base price. For example, suppose the base price was 1.87½ cents per pound, and the card extra for 24-inch I-beams was 0.10 cent. The price for 24-inch I-beams would be 1.87½+0.10=1.97½ cents per pound.

In cutting material in the mills, a ⅜-inch variation is allowed. That is, if you order a plate 18 feet 7½ inches long, it may come to you 18 feet 7⅞ inches long, or 18 feet 7⅛ inches long. So, in ordering material, it is ordered longer than the finished length, in order that it will be correct if the mills happen to cut it ⅜ inch shorter or longer than ordered. Of course, if it is longer than the ordered length, it can be cut again after it reaches the shops.

No shop work will be done on angles and plates at the mills, but they will do a certain amount on I-beams and channels. The work they will do on I-beams and channels is punching the same or different-sized holes in various parts, cutting and bending, and painting and fitting connection angles on the ends. It is often cheaper to have the mills do such work as the

above, and then have the material shipped directly to the site where the structure is to be located, thus saving some freight and all handling at the shops.

The mills issue cards stating the cost for performing the different operations as given above. This cost is in cents per pound of beam worked upon. It costs just as much to have one hole punched in the web of a 20-inch 65-pound I-beam 30 feet long, as it would to have 1,000 holes punched. A beam such as the above weighs $65 \times 30 = 1{,}950$ pounds; and it costs 15 cents per hundred pounds, or $19.5 \times 15 = \$2.93$, to punch as many holes as are wanted in the web of that beam. The card showing these extras and their cost may be had by addressing any rolling mill company. The cost varies from 15 cents to $1.55 per hundred pounds, according to the amount of work done.

67. **Drafting and Shop Costs.** The drafting cost on I-beams and channel work does not exceed 50 cents per ton when a good draftsman is employed on it. For a general average, $1.00 a ton is about right. Building-truss and plate-girder work should not run higher than $2.00 to $3.00 per ton, although some runs as high as $8.00 a ton. As a general rule, the lighter the work and the more difficult, the higher the cost. Some work on a dome where the heaviest angles were 3 inches by 3 inches by $^5/_{16}$ inch, and where much curved work was necessary, runs up as high as $15.00 a ton.

The Shop Cost. The cost of cutting, punching, assembling, and riveting, runs about as follows:

Column work	$14.00 to $20.00
Truss and Girders	$12.00 to $25.00

Other Costs. **Painting** will cost about from $1.50 to $3.00 per ton for two coats. **Bolts and spikes** are to be counted up, and estimated at the current market prices. **Freight** varies with the distance of the structure from the shop. **Erection** expenses are variable, depending upon the class of work, and must be figured separately for each case.

Traveling and **bidding** expenses vary with the distance of the agent from the place where the letting of the contract is done, when he is notified from the home office; and also with the character of the town in which the contract is let. Two days' hotel expenses is a fair amount for the estimate. **Haulage** may be estimated at 25 cents per ton-mile—that is, per ton hauled one mile.

68. **Profit.** This varies with the competition, and with the amount of work on hand in the shop. If work is plentiful, and men who wish employment are plentiful, the profit can be made low. If the shop is running full, and men are scarce, the profit can be made large, as it does not matter much if the contract is lost; and if it is awarded, then some of the other work may be delayed and the contract worked on.

On the other hand, if work is scarce, and the men are almost out of work in the shop, but prospects look brighter, it would be policy to take the work at a very small profit, or at none at all, or even at a small loss, in order to keep from laying off good men who might leave for other places and thus seriously inconvenience the output of the plant when work "picks up."

Ordinarily 15 per cent may be added for profit. If conditions are favorable during the contract, more than this may be realized. If adverse conditions occur, even this profit may vanish. The same estimator should confine his estimates to as few classes of work as possible, and success will be more certain.

ERECTION OF STRUCTURES

69. **The Marking Diagram.** Before the drawing of each piece of the structure leaves the drafting room, it is given a mark; and this mark is put on an outline drawing of the structure. In most cases, the letter which begins the name of the piece is put upon it, and this letter has a number which indicates which member it is. Thus columns are indicated C_1, C_2, C_3, etc.; beams B_1, B_2, etc.; girders G_1, G_2, etc.; and other members are designated in a similar manner.

In case of trusses, the lower joints are marked with the letter L with its subscript, and the upper joints have the letter U as in Fig. 87.

The great advantage of a marking diagram is that pieces are immediately recognized in the

stock pile. In the case of trusses, the notation used has an added value, since, by the mark on the member, it can be determined at once just where that member belongs. Thus L_1 L_2 is the second panel of the lower chord; U_2 U_3 is the third panel of the upper chord; U_1 L_1 is the first

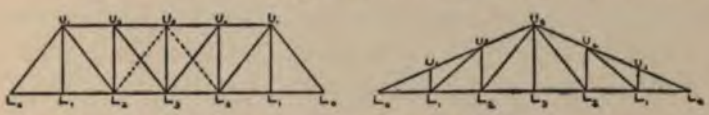


Fig. 87. Showing Notation Used in Marking of Trusses.

vertical; and L_2 U_3 is a diagonal in the third panel, as is also U_2L_3.

70. **Erection of Beams.** Beams are of such light weight that almost any light derrick is sufficient to raise them to position. Once in position, they are held there while the workmen connect them up to the other members with a few bolts, and they stay in this way until the riveters rivet them up.

71. **Erection of Girders.** Building girders are, if light, erected in the same manner as beams, a stiff-legged **derrick** usually being sufficient to raise them to place. In case one is insufficient, two may be used, one at each end. The girder may sometimes be more readily erected by a **gallows** or an **A-frame.** In such cases other parts of the same floor must have been erected previously, in order to provide supports for the gallows or A-frame, or these may have their supports on the floor below.

In railroad work four methods present them-

selves for consideration. These are:

(1) **Trestle falsework;**
(2) **Gin pole;**
(3) **Derrick cars or locomotive cranes;**
(4) **Derrick;**
(5) **Gallows frames.**

(1) In the first method, the **timber trestle work** is built up in a manner similar to that for trusses (Plate 24), and the track is laid out upon it. The girder is then either run out on a car, and skidded off the car into place; or it is unloaded by a gallows frame, or by a crane or derrick, and lowered into place (Plate 24).

(2) The **gin pole** method is applicable only to very small spans, and to locations where the surface of the ground is not far from the top of the rail. It is not much used.

(3) In case the structure is double-track, two **derrick cars** or two **locomotive cranes,** or one of each, may be used to lift the girder from one track into position on the other. Plate 25 shows a good example of this. In some cases the girder is picked up at the center and run out until the car nears the end of the track. The girder is then lowered into place. Plate 27 shows such a case.

(4) In cases where the stream is narrow or is dry, an ordinary **derrick** may be set up on the ground by the side of the track, and the girder erected as shown in Plate 26.

(5) Gallows frames are much used in the erection of girders. They are convenient and

economical, especially where they can be erected on the ground (Plate 24 and Fig. 88).

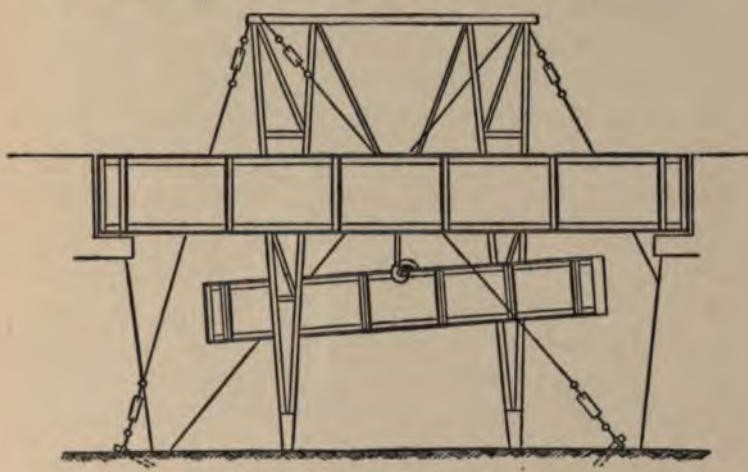


Fig. 88. Erecting Deck Plate-Girders with Gallows Frame.

72. **Erection of Simple Truss Bridges.** Simple truss bridges may be erected in four ways, namely:

(a) By gantry traveler and falsework;
(b) By derrick cars and falsework;
(c) By cantilever method, without falsework;
(d) By truss travelers.

(a) Plates 28 and 29 show good examples of the use of the **gantry traveler.** Piles are driven; and on top of these, timber bents are built. Upon the top of the bents are placed joists, and on these a floor of heavy timbers is placed. At the outer edges of the floor is a track; and along this track moves the gantry traveler,

which is nothing more than two gallows frames connected.

Several hoists are connected at the top of the gantry, and the various members which have been run out to the gantry on cars on the railroad track are taken and held in place while they are being connected. Plate 29 shows an end-post being held in place while the pin is being driven at the joint U_1.

(b) In case the gantry is not used, **derrick cars** may take its place (Plate 30).

(c) By the **cantilever** method, the two sides of a truss are erected at the same time (Fig. 89). No falsework is required, the halves of

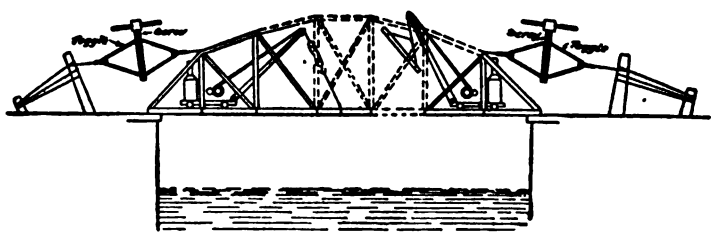


Fig. 89. Diagram Showing Erection with Toggle Joints and No Falsework.

the truss being kept in position by the stays at the ends. When the center is reached, the stays are lengthened or shortened by the turnbuckles, and the span goes together, and is connected up.

(d) In some instances where a number of spans of equal length are to be built, it is economical to use a **truss traveler,** as shown in Plate 31. When one span is begun, the traveler runs out until the end comes over the next pier

Courtesy of R. E. Gaut, Bridge Engineer, I. C. R. R.

ERECTING A VIADUCT OVER BRUSH CREEK, ALABAMA.

On Birmingham extension of Illinois Central Railroad. Height of viaduct, 185 ft. View showing use of cantilever traveler.

Courtesy of R. E. Gaut, Bridge Engineer, I. C. R. R.

ERECTING A VIADUCT OVER BRUSH CREEK, ALABAMA.

On Birmingham extension of Illinois Central Railroad. Height of viaduct, 185 ft. View showing use of cantilever traveler.

PLATE 34—STEEL CONSTRUCTION.

ahead, and then the vertical posts are put under it. The truss is then erected piece by piece, and is held up by being hung to the traveler. When the last number is in position, the span is let down on the masonry, and the traveler moves up ready for the next span. No timber falsework is required in such cases.

In the case of rolling-lift bridges, the erection is usually made when they are open, the traveler being so constructed as to move up the top chord (see Plate 32). Sometimes they are erected when closed, and this is done by any of the methods described above for simple non-movable bridges.

73. **Erection of Viaducts.** This is best done by the use of either a derrick car or a **cantilever traveler** (Plate 33). This method is fully illustrated in Plates 34 and 35. In erecting by these methods, care should be taken to see that the anchor-bolts are in the girders; otherwise the end girder is liable to be shoved forward by the action of the trains and travelers, and will fall from the masonry. One advantage of the cantilever traveler over the derrick car is that it has a greater number of hoists, and can therefore hold members in any desired position much better than the derrick car, which depends upon the balancing of the member.

74. **Erection of Mill Buildings.** Here the columns are first erected by means of a derrick, as in Plate 26, and are securely bolted to the masonry. In case the span of truss is small, the

truss is completed before the erection; and the same derrick will raise the span into position and the workmen will bolt it up for the riveters. Where the span is great, falsework, as in the case of bridges, may be employed and travelers are used, the truss being erected a piece at a time (see Plate 36).

75. **Erection of Office Buildings.** The erection of office buildings is simpler than that of any other class, except mill buildings. The columns are erected in two-story lengths; and, after the girders and floor-beams of one floor are put in place and connected up, the erecting derricks are moved up another floor. Unless the building is large, one derrick will be sufficient for the erection. Plates 21, 22, 37, and 38 show very clearly the methods of erection.

BUILDING SUPERINTENDENCE

76. **Shipping Clerk and Yard Master.** The business of the **Shipping Clerk** should be to keep track of the shipping bills, and to see what material comes in. He should at once notify the **Yard Master** and **Storekeeper,** in order that they may provide for it.

The **Yard Master's** position is very important; and upon him, as much as on anyone, depends whether or not the work will progress without a hitch. He should be familiar with the marking diagram. He should keep all material of one class—say all diagonals, all posts, all top chords, etc.—in separate piles easy of access,

and should have them piled up so that their marks can easily be found and read. He should be on hand constantly, in order to see that the erection gang may find a member immediately. A carefully kept account of all material received, and all given out, and to whom given, will greatly simplify matters in case of a misunderstanding, and will also indicate what members have been erected and what tonnage has been erected in a given time.

77. **Rivet Foreman.** A general **Foreman of Riveters** is important to the efficient working of this class of labor. His duty should be to see that a plentiful supply of fuel, rivets of the proper size and length, and tools, is furnished each rivet gang. He should also keep an eye on the air-hose and the compressor plant, and be familiar with them. Above all, he should himself be a practical riveter, and should not hesitate to instruct his men in case they show the need. If it is known by the rivet gangs that the Foreman will not stand for burnt rivets when the Inspector is not present, a great amount of expense will be avoided, since a good Inspector will detect the rivets, the result being that they have to be cut out, the hole rebored in many cases, and a larger rivet driven.

78. **Foreman of Riggers.** Many firms let their erection men do the rigging; but on work of any magnitude—and indeed on any work—it is better to have a foreman and gang who do nothing but the rigging, and keep the material

in order. This latter point is important, since a breaking rope will ofttimes cause a member to be so bent as to require that it be returned to the shop to be fixed up; and in this manner a very large part of the prospective profit will vanish. Good riggers, with their knowledge of knots and splices, are a valuable adjunct for the economical erection of structures.

79. **Foremen of the Erection Gangs.** On small jobs, the Superintendent is the Foreman; and on many other jobs, one foreman is all that is required. On large jobs, or where erection is from the two ends toward the center, it is advisable to have two foremen. In such cases, they should report directly to the Superintendent; and men working under one should, if possible, be kept under him, since a man used to the methods of one foreman will likely find fault with the methods of the other, and this will tend to lessen the efficiency of the organization. The Foreman should see that the next piece in order is ready before the men are ready for it, so that it can be lowered into place without delay; and should take care to see that no man wastes time trying to do something that could be done in a few minutes with a block and tackle and the assistance of the hoisting engine.

80. **Efficiency of Working Force.** The efficiency of the working force may be increased in several ways. The first is to stimulate interest in the work by the offering of a **bonus**, which of course is given by the firm. Determine the aver-

age tonnage erected, and the average number of rivets driven; and then employ a boy to keep track of these, and post them on a blackboard at the end of the day, so that all may see what the different gangs have done. Watch the increase in the amount of work done! If the company will not offer a bonus, give the best gang for that week a half-day or a few hours off. The result will be surprising, as each will try to outdo the other. This has the additional advantage of cutting out the poor man, since the better men in the gang do not want him with them, no matter how "good a fellow" he may be.

If possible, begin to erect from both ends. The erection gangs will race to see who reaches the middle first, if they are told that the one getting to the middle first may have a holiday with pay, or a bonus.

If one class of erection is pleasanter than another, give the highest tonnage gang the pleasantest work at the end of each week. Watch them work to stay there!

Other methods will suggest themselves in each special case, and the Superintendent's reward will come as an increase in salary or a more important job.

81. **Reports.** Reports of progress on the work are made to the home office at intervals of a week or a month. They should contain a statement of the supplies and tonnage on hand, the tonnage erected, the number of men working, and the number of days worked. They should be

accompanied by photographs of the work. These photographs should be taken from several points of view, and always so as to show the progress made during the period from one report to another. They should be neat, and all sketches accompanying them should be neat. A little red ink underlining the headings will improve the looks of a report to a surprising degree. In addition to the above-mentioned information, any additional facts which you think would interest the home office should be put in.

82. **Specifications.** This is here briefly referred to, since it is the duty of the Superintendent of Erection to see that none of the clauses of the specifications are violated. He should make himself familiar with them. Where the specifications are short, this is easily done. Where they are long, it is almost impossible to do so all at once. In this case, the Superintendent should be familiar with that part which refers to the work under way at any particular time.

Perhaps it would be well to give rules for success, rules which have been given by two as successful engineers as the world has seen. These rules are:

(1) "The most perfect system of rules to insure success must be interpreted upon the broad ground of professional intelligence and common sense."

(2) "**Work, work, work!**"

DINING ROOM WITH PENDANT ELECTRIC LIGHT FIXTURE.

Electric Wiring

There is no application of electricity in the arts and industries that has received more attention and made greater advancements in the last quarter of a century, than that made in the field of Electric Lighting. Twenty to thirty years ago, there were no such things as electric lights except those shown in laboratory or lecture-room experiments. The candle, oil lamp, and gas were the only means of artificial illumination, and people were satisfied with the moderate amount of light obtained from these sources.

With the marked improvement in the electric lamp, and the decrease in cost of electrical energy, we find countless thousands of electric lights in use at the present time for all purposes of illumination. Indeed, so important a factor has the electric current become in the domestic, business, and industrial life of our times, that a knowledge of its fundamental principles and the approved methods of its application to various uses is to-day a necessity to everyone who is engaged in or interested in building construction of any sort.

Even in locations where electricity is not yet available, it is becoming the universal practice

in the erection of buildings of all sorts, to provide for the future by wiring them for electricity. And if for no other object than to be able to direct the work of wiring intelligently, or to make the repairs which will occasionally be necessary, or to install minor conveniences such as bells, annunciators, burglar alarms, etc., which may be placed in any house whether fully wired or not, a knowledge of the principles of electric wiring and of the materials and fixtures used will serve a very useful purpose. In the following pages will be found all the essentials required by those who wish to wire their homes for electric light, or who expect to have the same done by an electrical contractor.

Wiring For Illumination

The problem of wiring a house for electric lights, in its broader sense, does not mean the mere placing of electric wires and making the proper connections; but one must have a knowledge of the illuminating properties of different lamps and the factors influencing such illumination—namely, distribution of light from lamp, type of reflectors, and location and number of lamps. In addition, one must be able to calculate size of conductors required to safely carry given current, with a certain percentage loss.

On account of the above requirements it is necessary to make a brief study of the electrical circuit, the quantities involved and their relation to one another, also a few illumination calculations, before proceeding to the actual problem of wiring.

THE ELECTRICAL CIRCUIT

The electrical circuit can best be studied by making a comparison between it and a hydraulic problem where the various quantities bear practically the same relation to one another. When there is a flow of water through a pipe from a pump or some source of pressure, the stream of water flowing depends upon the pressure and upon the resistance that the pipe offers in the way of friction—it is, in fact, proportional to the **pressure** divided by the **frictional resistance.**

The same condition exists in an electrical circuit, where a current of electricity flows in a wire due to the pressure of a battery or other source of electrical energy; and the value of the current is equal to the **pressure acting,** divided by the opposition or **resistance** offered by the circuit to such flow.

When the electrical pressure acting tends to send a current through the circuit always in the same direction, it results in what is termed a **direct current;** if, however, it is continuously reversing in direction—first in one direction and then in the opposite direction—the current flowing then will be what is termed an **alternating current.** The following discussion applies only to direct current, which is the form of current commonly used in houses for lighting, heating, etc.

In the hydraulic problem, the resistance offered to the flow of water, neglecting that due to bends, depends upon the length, the area, and the condition of the inner surface of the pipe. If the length of the pipe is increased, resistance is increased; if the area is increased, resistance is decreased; if the inner surface of the pipe is smooth, it will offer less resistance than it would if rough. There is a somewhat similar condition in the electrical circuit. If the area of a conductor carrying current remains constant, resistance will increase directly as the length of the conductor increases; if area increases, resistance will decrease at the same rate.

Resistance will also depend upon the kind of material that forms the circuit, which corresponds to a certain extent to condition of surface of pipe. The resistance, in the case of pipe, is only on the outer surface of the stream in contact with the pipe; while, in the case of the electrical circuit, it is uniform throughout the conductor. The resistance of a circular conductor .001 of an inch in diameter (one **mil**) and one foot long, is termed the **mil-foot resistance** of the material. If this value is known, which is 9.52 ohms for copper at a temperature of 0° Centigrade (the freezing point of water), the value of the resistance of a copper conductor of any given dimensions can be calculated by the use of the following equation:

$$R = 9.52 \times \frac{\text{Length in feet}}{\text{Area in circular mils}}.$$

Area in circular mils equals diameter in mils squared. This means that the resistance is found by multiplying the length of the conductor by 9.52, and dividing the product by the area of the conductor in circular mils.

Table I gives the diameters, weights, and resistance of copper wires drawn according to the Brown & Sharpe gauge.

The **conductivity** of a conductor, or its power of receiving and transmitting current, bears an inverse ratio to its resistance. Copper, on account of its commercial application as an elec-

TABLE I

Properties of Copper Wire

English System—Brown & Sharpe (B. & S.) Gauge

Nos.	Diameter in Mils	Area in Circular Mils	Weights in Pounds		Resistance per 1,000 Feet in Ohms	
			1,000 ft.	Mile	At 60° F.	At 75° F.
0000	460	211,600	641	3,382	.04811	.04966
000	410	168,100	509	2,687	.06056	.06251
00	365	133,225	403	2,129	.07642	.07887
0	325	105,625	320	1,688	.09639	.09948
1	289	83,521	253	1,335	.1219	.1258
2	258	66,564	202	1,064	.1529	.1579
3	229	52,441	159	838	.1941	.2004
4	204	41,616	126	665	.2446	.2525
5	182	33,124	100	529	.3074	.3172
6	162	26,244	79	419	.3879	.4004
7	144	20,736	63	331	.491	.5067
8	128	16,384	50	262	.6214	.6413
9	114	12,996	39	208	.7834	.8085
10	102	10,404	32	166	.9785	1.01
11	91	8,281	25	132	1.229	1.269
12	81	6,561	20	105	1.552	1.601
13	72	5,184	15.7	83	1.964	2.027
14	64	4,096	12.4	65	2.485	2.565
15	57	3,249	9.8	52	3.133	3.234
16	51	2,601	7.9	42	3.914	4.04
17	45	2,025	6.1	32	5.028	5.189
18	40	1,600	4.8	25.6	6.363	6.567

trical conductor, is taken as having a 100 per cent conductivity; and all other materials are compared with it as a standard.

Electrical Units and Definitions

It might be well at this point to give the names of the various electrical quantities and their relations.

The unit in which we measure electrical pressure is the **volt,** the ordinary dry cell giving approximately 1.5 volts.

The unit in which current is measured is the **ampere,** and the unit of resistance is called the **ohm.**

$$\text{Current} = \frac{\text{Voltage}}{\text{Resistance}}$$

$$\text{Amperes} = \frac{\text{Volts}}{\text{Ohms}}$$

The power used in causing a current to flow in a circuit is measured in a unit called the **watt;** and the number of watts expended in a circuit is equal to electrical pressure at the terminals, multiplied by the current flowing; that is: Power = Volts × Amperes. A thousand watts is called a **kilowatt.** There are 746 watts to the **horse-power.** A watt acting for one hour is called a **watt-hour,** and is a unit of electrical energy. A **kilowatt-hour** is a thousand watts for one hour, or any number of watts which, multiplied by the time they act in hours, gives 1,000. The kilowatt-hour is the unit used in selling electrical energy. The wattmeter is so constructed that its reading is proportional to the voltage multiplied by the current multiplied by the time in hours multiplied by some quantity (known as **constant of wattmeter**), which is often unity.

Series and Parallel Circuits

When an electrical circuit is composed of a number of different parts, such as a bell, battery, push-button, and conductors connected as shown in Fig. 1, so that current flows in a single path, it is called a **series circuit.** The total resistance of the circuit is equal to the sum of the resist-

ances of the various parts. Suppose the bell has a resistance of 10 ohms, and the conductors a resistance of 2 ohms, then the total resistance of the circuit would be 10+2=12 ohms. The current in all parts of the circuit is the same.

If the current in flowing from one point in a circuit to another has more than one path, we have what is called a **parallel circuit.** The points A and B in Fig. 2 are connected by two paths,

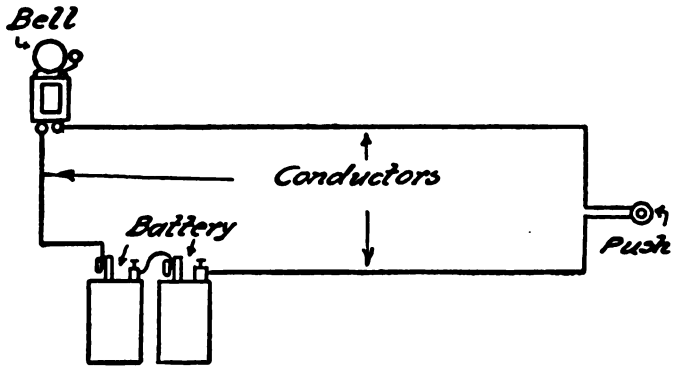


Fig. 1. Diagrammatic Representation of a Series Circuit.
Current flowing in a single path.

one through lamp I, and another through lamp II. It is apparent that the resistance between two points is lessened as we increase the number of paths. If the resistances of all the paths are equal, the combined resistance is equal to the resistance of any path, divided by the number of paths. If the resistances are unequal, the total resistance must be calculated from following equation:

$$\frac{1}{R}=\frac{1}{R_1}+\frac{1}{R_2}+\frac{1}{R_3}+\text{etc.},$$

in which R equals the combined resistance; and R_1, R_2, R_3, etc., equal the resistances of various branches. This equation means that if you add together the quotients obtained by dividing

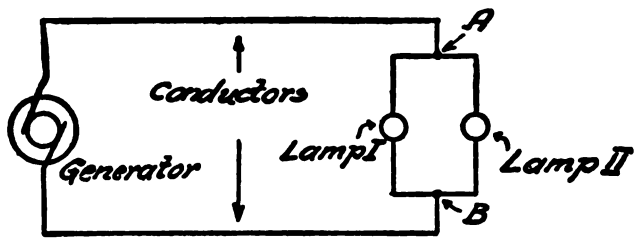
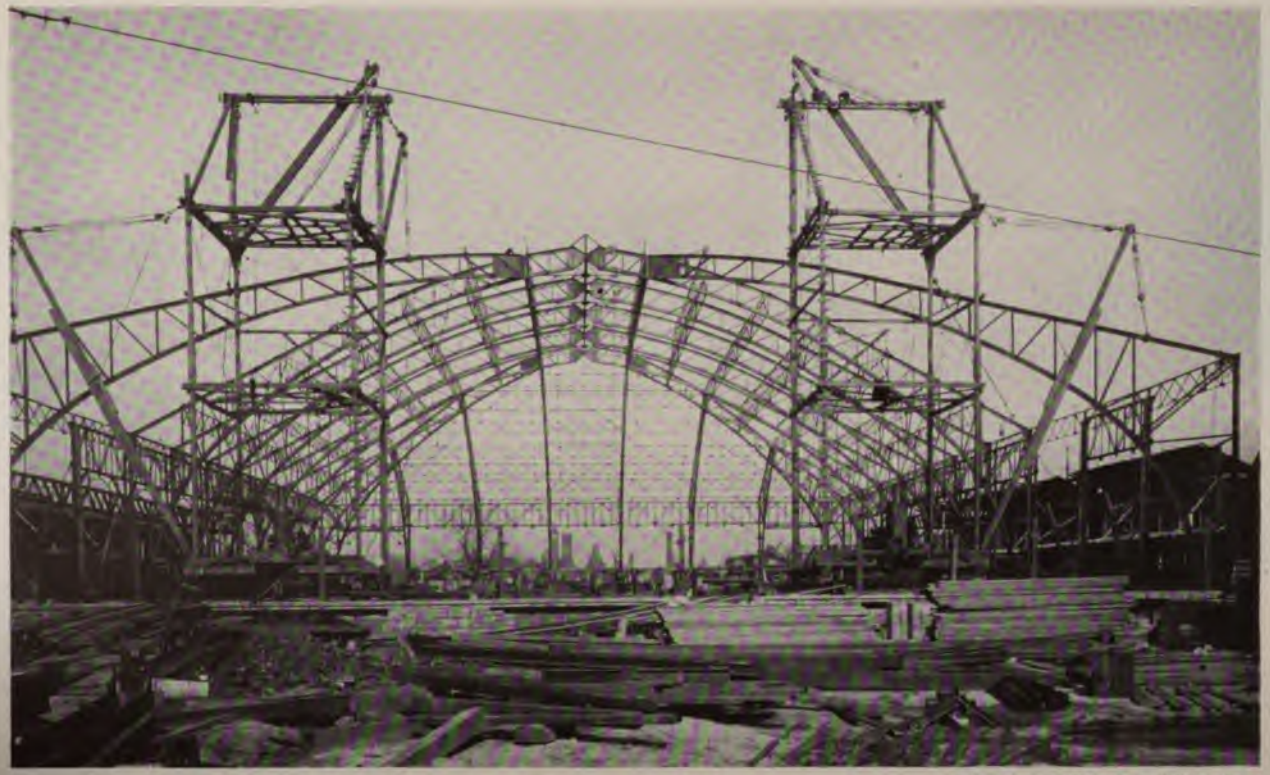


Fig. 2. Diagrammatic Representation of a Parallel Circuit. Current flowing in more than one path.

unity by the various resistances, and then divide the sum into unity, it will give you the combined resistance.

The current in the main circuit is equal to the sum of the currents in the various branches. The currents in the branches are equal if the resistances are equal; otherwise, currents vary inversely as resistances; that is, as resistance diminishes, current increases, and vice versa.

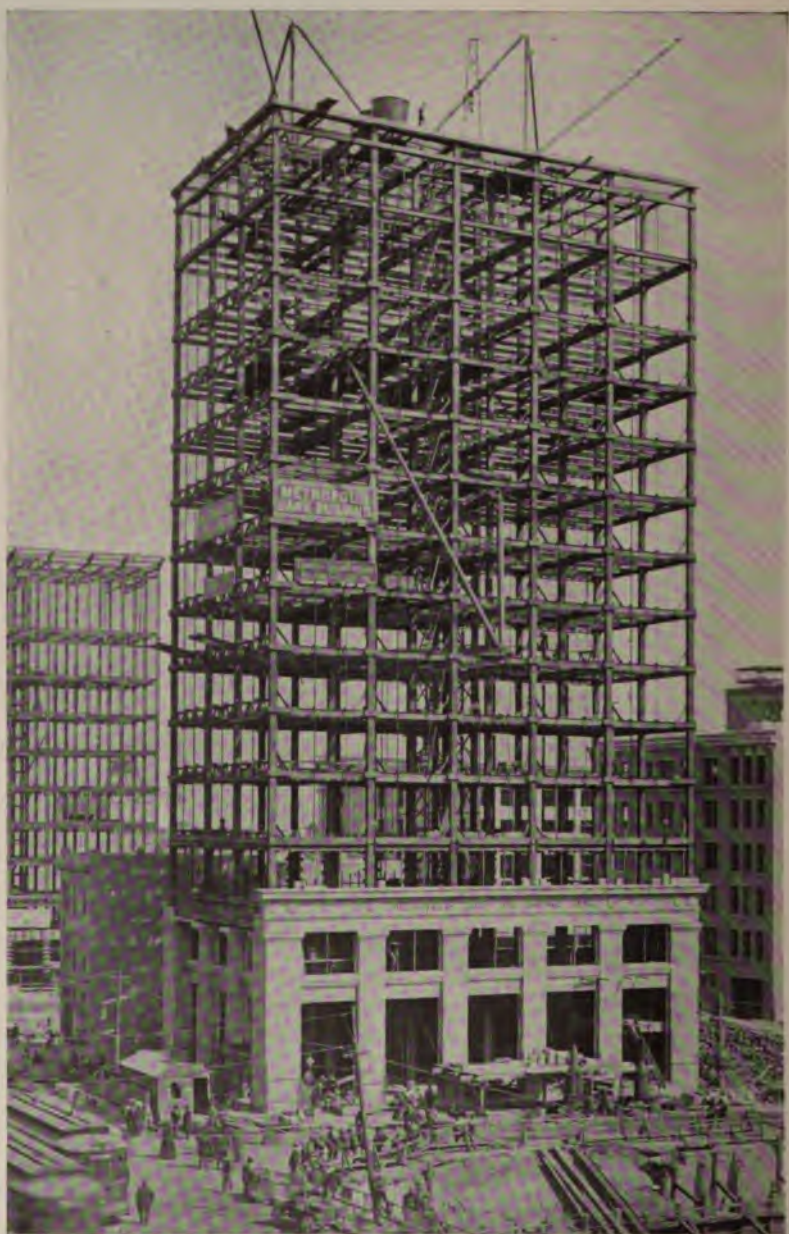
Let us assume a condition such as shown in Fig. 3, where there are a number of lamps arranged to be connected to a switch S through conductors C_1 and C_2. If the voltage at the switch is maintained constant at say 110 volts, the following conditions will exist as lamps are turned on: Before any lamps are turned on, the



Courtesy of American Bridge Co.

STEEL FRAME FOR 65TH REGIMENT ARMORY, BUFFALO, N. Y.

Dimensions of building, 240 ft. by 336 ft. Weight of steel, 2,980,000 lbs.



Courtesy of American Bridge Co.

METROPOLITAN TRUST & SAVINGS BANK, SAN FRANCISCO, CAL., UNDER CONSTRUCTION.

Weight of steel, 1,600,000 lbs.

PLATE 37—STEEL CONSTRUCTION.

may be obtained by adding them. Then, assuming a certain loss in the line, say 2 per cent, and knowing the total length of conductor forming the circuit, we can calculate the size of wire required.

Suppose it is desired to conduct 10 amperes 100 feet with a loss of 2 per cent, the voltage of the line being 110. The total length of the circuit is 200 feet; the drop in voltage in the line is 2 per cent of 110=2.2 volts. Resistance=2.2÷10=.22 ohm. If .22 ohm is the resistance of 200 feet, then 1,000 feet, being five times as long, would have a resistance of .22×5=1.1 ohms. Referring to Table I, we find that a Number 10 wire has a resistance of .9785 ohm per 1,000 feet, which is next below this value. Tables II and III give sizes of wires to transmit a current of from 1 to 100 amperes in value over a conductor from 40 to 400 feet in length, with a 2 per cent loss.

The size of wire obtained from Tables II and III may conduct current with a 2 per cent loss; but its current capacity may not be sufficient. No wire smaller than a Number 14 shall be used, except for fixtures.

Table IV gives the current capacity for different sizes for both open and concealed work.

If the size of wire calculated from Tables II and III is smaller than that allowed in Table IV for a given current, the size in Table IV should then be used.

TABLE II

Sizes of Wire for 110-Volt Circuit with 2 per Cent Loss

No. Amperes	Length of Conductor in Feet																		
	40	60	80	100	120	140	160	180	200	220	240	260	280	300	320	340	360	380	400
1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	14	14
1.5	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	14	14	13	12	12
2	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	14	14	13	12	12	11	11
3	.....	.....	.....	.....	.....	.....	.....	.....	.....	14	14	13	12	12	11	11	10	9	9
4	.....	.....	.....	.....	.....	.....	.....	14	14	13	12	12	11	11	10	9	9	8	8
5	.....	.....	.....	.....	.....	14	14	13	13	12	11	11	10	10	9	8	8	7	7
6	.....	.....	.....	.....	14	14	13	12	12	11	11	10	9	9	8	8	7	7	6
7	.....	.....	.....	14	14	13	12	12	11	11	10	9	9	8	7	7	6	6	5
8	.....	.....	.....	14	13	12	12	11	11	10	9	9	8	8	7	7	6	5	5
9	.....	.....	14	13	12	12	11	11	10	9	9	8	8	7	7	6	5	5	4
10	.....	.....	14	13	12	11	11	10	10	9	8	8	7	7	6	5	5	4	4
12	.....	14	13	12	11	11	10	9	9	8	8	7	7	6	5	5	4	4	3
14	.....	14	12	11	11	10	9	9	8	7	7	6	6	5	5	4	3	3	2
16	.....	13	12	11	10	9	9	8	8	7	7	6	5	5	4	3	3	2	2
18	14	12	11	10	9	9	8	8	7	7	6	5	5	4	4	3	2	2	1
20	14	12	11	10	9	8	8	7	7	6	5	5	4	4	3	2	2	1	1
25	13	11	10	9	8	7	7	6	6	5	4	4	3	3	2	1	1	0	0
30	12	10	9	8	7	7	6	6	5	4	4	3	3	2	1	1	0	0	00
35	11	10	8	7	7	6	5	5	4	4	3	2	2	1	1	0	00	00	000
40	11	9	8	7	6	5	5	4	4	3	2	2	1	1	0	00	00	000	000
45	10	9	7	6	6	5	4	4	3	3	2	1	1	0	00	00	000	000	0000
50	10	8	7	6	5	4	4	3	3	2	1	1	0	0	00	000	000	0000	0000
60	9	7	6	5	4	4	3	3	2	1	1	0	0	00	000	000	0000	0000	.....
70	8	7	5	4	4	3	2	2	1	1	0	00	00	000	000	0000	0000	.....	.....
80	8	6	5	4	3	2	2	1	1	0	00	00	000	000	0000	0000	.....	.....	.....
90	7	6	4	3	3	2	1	1	0	00	00	000	000	0000	0000	.....	.....	.....	.....
100	7	5	4	3	2	1	1	0	0	00	000	000	0000	0000	.....	.....	.....	.....	.....

TABLE III

Sizes of Wire for 220-Volt Circuit with 2 per Cent Loss

No. Amperes	Length of Conductor in Feet																		
	40	60	80	100	120	140	160	180	200	220	240	260	280	300	320	340	360	380	400
1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	14
1.5	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	14	14
2	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	14	14
3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	14	14	13	12	12
4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	14	14	13	12	12	11	11
5	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	14	14	13	13	12	11	11	10	10
6	.....	.....	.....	.....	.....	.....	.....	.....	.....	14	14	13	12	12	11	11	10	9	9
7	.....	.....	.....	.....	.....	.....	.....	14	14	14	13	12	12	11	11	10	9	9	8
8	.....	.....	.....	.....	.....	.....	.....	14	14	13	12	12	11	11	10	9	9	8	8
9	.....	.....	.....	.....	.....	.....	14	14	13	12	12	11	11	10	9	9	8	8	7
10	.....	.....	.....	.....	.....	14	14	13	13	12	11	11	10	10	9	8	8	7	7
12	.....	.....	.....	.....	14	14	13	12	12	11	11	10	9	9	8	8	7	7	6
14	.....	.....	.....	14	14	13	12	12	11	11	10	9	9	8	7	7	6	6	6
16	.....	.....	.....	14	13	12	12	11	11	10	9	9	8	8	7	7	6	6	5
18	.....	.....	14	13	12	22	11	11	10	9	9	8	8	7	7	6	5	5	4
20	.....	.....	14	12	12	11	11	10	10	9	8	8	7	6	6	5	5	4	4
25	.....	14	13	12	11	10	10	9	9	8	7	7	6	6	5	4	4	3	3
30	.....	13	12	11	10	10	9	9	8	7	7	6	6	5	4	4	3	3	2
35	14	13	11	10	10	9	8	8	7	7	6	6	5	5	4	4	3	2	1
40	14	12	11	10	9	8	8	7	7	6	6	5	5	4	4	3	2	2	1
45	13	12	10	9	9	8	7	7	6	6	5	4	4	4	3	3	2	1	1
50	13	11	10	9	8	7	7	6	6	5	4	4	4	3	3	2	1	1	0
60	12	10	9	8	7	7	6	6	5	4	4	3	3	2	1	1	0	0	00
70	11	10	8	7	7	6	5	5	4	4	3	2	2	1	1	0	00	00	000
80	11	9	8	7	6	5	5	4	4	3	2	2	1	1	0	00	00	000	000
90	10	9	7	6	6	5	4	4	3	3	2	1	1	0	00	00	000	000	0000
100	10	8	7	6	5	4	4	3	3	2	1	1	0	0	00	000	000	0000	0000
120	9	7	6	5	4	4	3	3	2	1	1	0	0	00	000	000	0000	0000	.....

TABLE IV

Current Capacities of Wires for Open and Concealed Work

No., B. & S. Gauge	Concealed Work	Open Work
14	12 amperes	19 amperes
12	17 "	24 "
10	24 "	32 "
8	33 "	43 "
6	46 "	57 "
5	54 "	68 "
4	65 "	74 "
3	76 "	88 "
2	90 "	98 "
1	107 "	117 "
0	127 "	140 "
00	150 "	157 "
000	177 "	185 "
0000	210 "	225 "

Systems of Distribution

There are a number of systems employed in distributing electrical energy for lighting purposes, the most important of which are the following:

Parallel System. The parallel system in its simplest form is shown in Fig. 3. All the lamps are connected to the same two wires, and the current through each lamp is independent of the current through any of the other lamps. The length of conductors used in calculating the sizes for the wires would be twice the distance from the switch to point D, the most central point in the group of lamps. When this system is increased in size, it becomes more complex, as shown in Fig. 4. In this case there are branch circuits led off from main line to various parallel circuits. The length of conductor used in calculating the size of wire for the main line is taken as the distance from switch S to point D,

which is termed **Center of Distribution** of system. The calculation for various branch circuits is the same as that for Fig. 3.

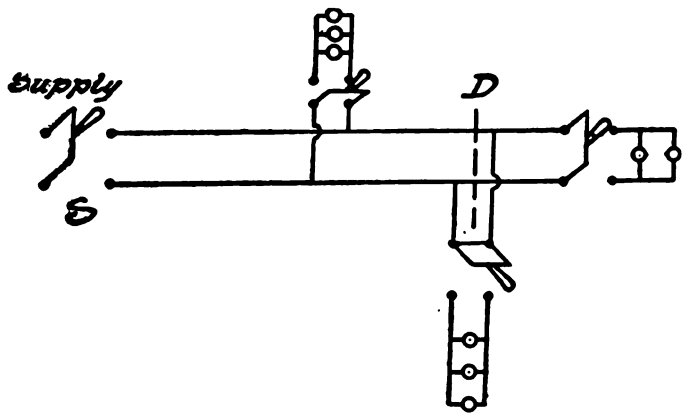


Fig. 4. Simplest Form of Parallel System of Distribution.

Multiple-Series System. The multiple-series system is shown in Fig. 5. The lamps are connected in groups of two or more in series, depending upon voltage between lines and voltage required to operate lamps, and these groups are connected in parallel. The total voltage between lines is distributed over the lamps in series. For example, if the voltage of the line is 220, and there are two lamps of equal resistance in series connected to it, each lamp will have a difference of potential between its terminals, of 110. This is the system employed in lighting street-cars where the voltage is approximately 550, five lamps being connected in series and constituting a single group. In this

system it is impossible to have a single lamp operate by itself, as the current through any one lamp must pass through all the rest of the lamps of the same group.

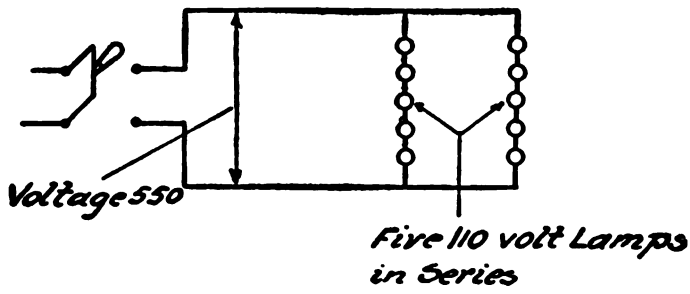


Fig. 5. Multiple-Series System of Distribution.

Three-Wire System. The three-wire system of distribution is shown in Fig. 6, which con-

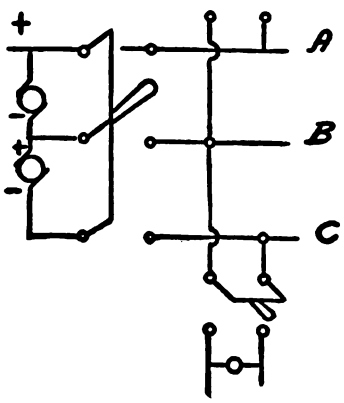


Fig. 6. Three-Wire System of Distribution.

sists of two equal voltages connected in series. At the junction of the two machines, a third wire is run, which is negative with respect to A,

and positive with respect to C. If lamps are connected to A and B, the voltage across them will be that of machine I, and current will flow on line A and return on line B. If, now, lamps are changed from A and B to B and C, the voltage across them will be that of machine II, and current will flow out on B and return on C. It is seen that the current in C is away from the machines in one case, and toward them in the other. Hence, if the same number of lamps are turned on in both cases, the current in B will be zero, since the two currents neutralize each other. The output of this system is equivalent to two independent circuits, there being quite a saving in copper.

ILLUMINATION CALCULATIONS

Incandescent lamps are usually designated as being of so many **candle-power**, meaning they give a certain number of times the light given by a standard candle of definite dimensions and burning at a certain rate. The efficiency of a lamp is based upon the relation of the candle-power output to the watts input. The ordinary 16-candle-power **carbon filament lamp** requires an input of approximately 56 watts, or it has an efficiency of 3.5 watts per candle. The **tungsten lamp** has an efficiency of approximately 1.25 watts per candle; and the **tantalum lamp**, approximately 2.0 watts per candle. The candle-power of a lamp is not the same in all directions, but depends upon the construction of the lamp.

After it is installed, candle-power in different directions will depend upon the position of the lamp, and upon the kind of reflector or shade

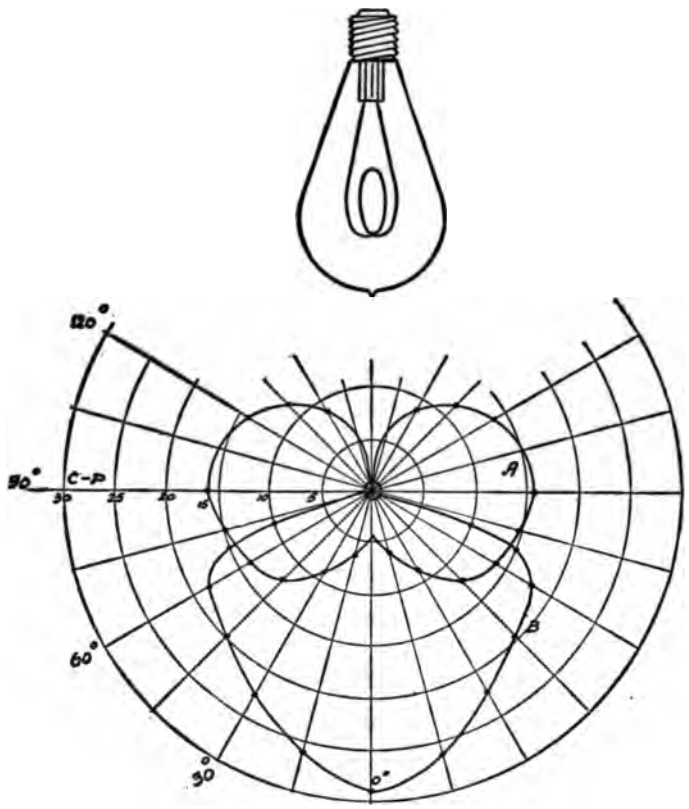


Fig. 7. Illumination Diagram.

used. In Fig. 7, the length of line from O to curve A in any direction, is proportional to the candle-power of lamp in that direction. Curve B gives the distribution of light when an ordinary tin shade is used, the inner surface being enameled.

Assuming that we know the general form of such curves for various lamps and reflectors, we can now calculate the candle-power required to give a certain illumination. The illumination on any surface is measured in a unit called the **foot-candle**, it being the light thrown on a surface one foot from a standard candle. Its value will vary directly as the candle-power of the source of light, and inversely as the square of the distance from same. This means that a 16-candle-power lamp will give twice as much illumination as an 8-candle-power lamp, 4 times as much as a 4-candle-power lamp, etc.; also, that if a lamp gives a certain illumination at 1 foot distance, it will give only one-fourth as much at twice the distance, one-ninth as much at 3 times the distance, one-sixteenth as much at 4 times the distance, etc.

In Fig. 7, the candle-power along the 45-degree line from curve B is 19. The illumination 3 feet from the lamp would be $19 \div 3^2 = 2\ 1/9$ foot-candles.

The approximate values of the illumination for various parts of a residence are as follows:

General illumination		1.0—2.0	foot-candles.
Reading	Clear print	1.0—1.5	" "
	Newspaper print	2.0—2.5	" "
	Reading tables	3.0—4.0	" "
Ball rooms		2.0—3	" "

Knowing the required illumination, the candle-power required can be calculated.

CHOICE OF LAMPS

The choice of lamps to use will depend upon initial cost of lamp, cost of electrical energy, fluctuations in voltage applied, and treatment of lamp after installed.

If the price of electrical energy is low, it may be best to use a low efficiency lamp, especially if fluctuations in voltage are very marked. The tantalum and tungsten lamps are much more fragile than carbon filament lamps, and the cost of renewals would more than offset the gain in efficiency where the breakage is large.

The location of the lamps will in a measure depend upon whether they are to give a general illumination, or whether the light is to be directed to certain areas such as reading tables, etc.

FIXTURES, OUTLETS, SWITCHES, ETC.

In residence wiring, we are as a rule confined to **chandeliers, wall brackets,** and **ceiling lights.** The wall brackets are usually employed to increase the general illumination at certain points. Often **outlets** are provided in baseboards, where portable lamps may be connected. In locating lamps and outlets, it is well to bear in mind that each outlet means an additional cost in the wiring.

After the outlets have been located, we next pass to the location of **switches, distributing board, cut-out boxes, service switch,** and **meters.**

The switches controlling the various groups of lamps should be located at the most convenient point in the room, which is usually, not always, near the entrance, so that one may turn the lights on or off just as he enters or leaves the room. The distributing board is, as its name implies, a distributing point where various branch circuits are connected to the main line. It should be located so as to make the various branch circuits as near the same length as possible.

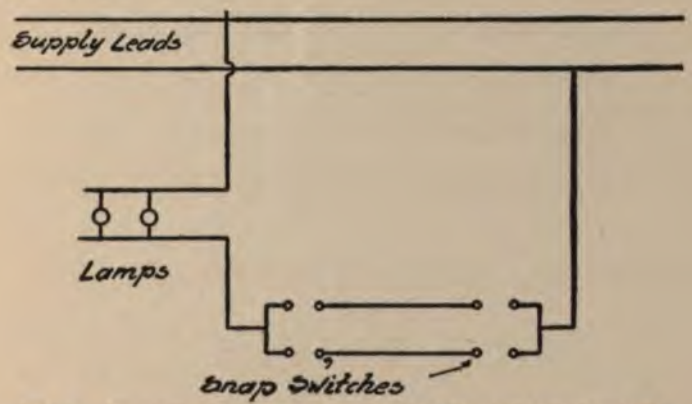


Fig. 8. Arrangement of Two Switches to Control Same Circuit.

Quite often, two switches are arranged to control the same circuit (see Fig. 8), being interconnected in such a way that lamps can be turned on or off at either switch independently of the other. This is quite a great convenience in long hallways, and also in being able to control lamps in stairways from a switch on either floor.

Fig. 9 shows a two-wire distributing panel;

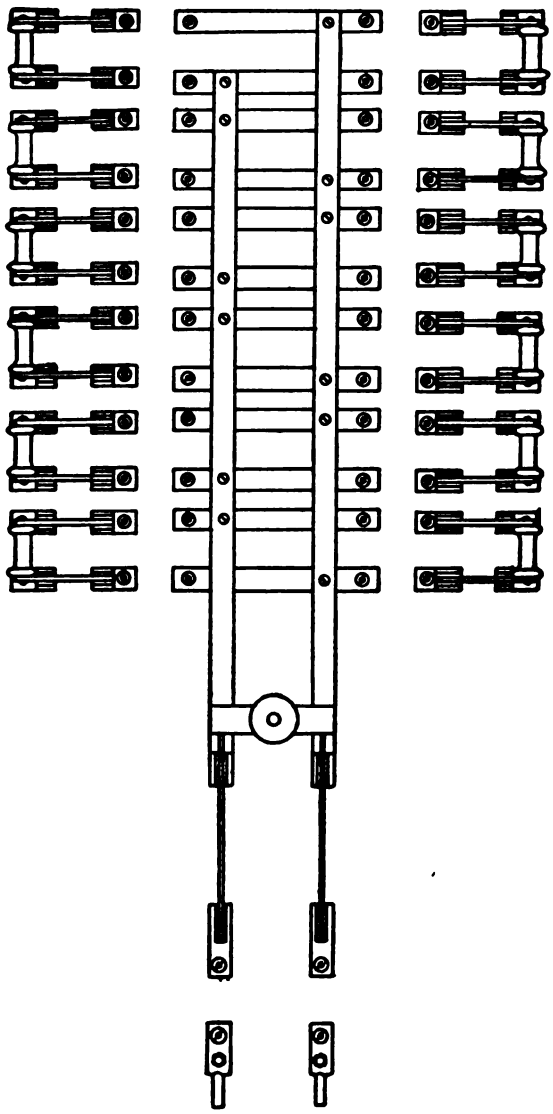


Fig. 9. Two-Wire Distributing Panel.

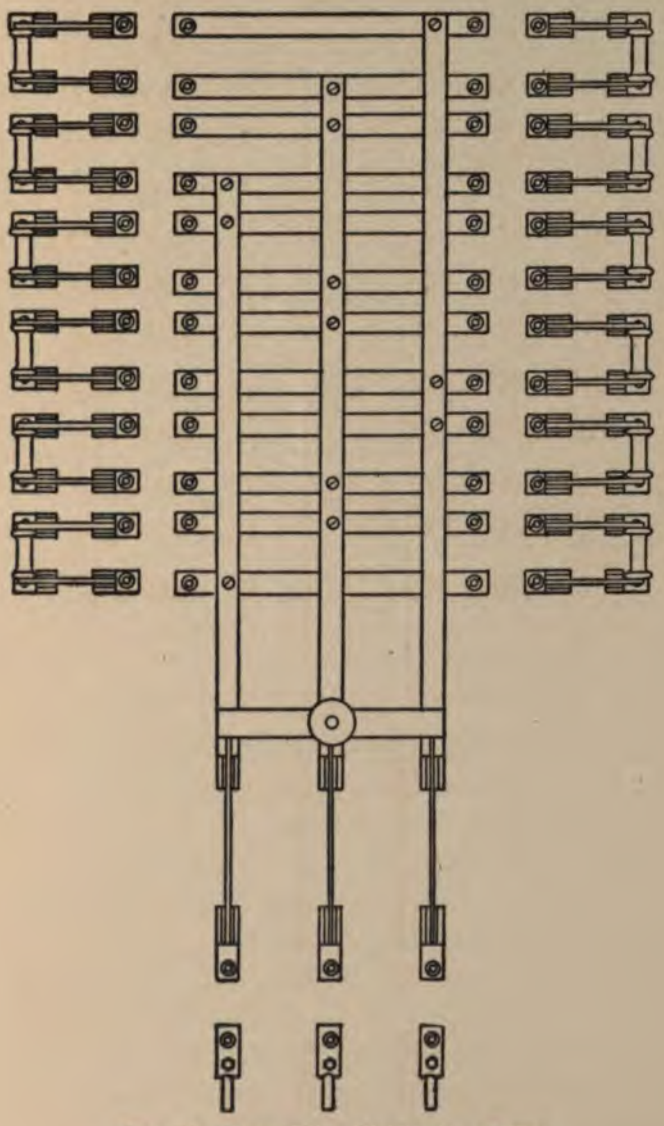


Fig. 10. Three-Wire Distributing Panel.

and Fig. 10 a three-wire. Each of the branch circuits is provided with a fuse protection, and should not deliver more than 660 watts to any group of lamps. These distributing panels should be enclosed in fireproof boxes provided

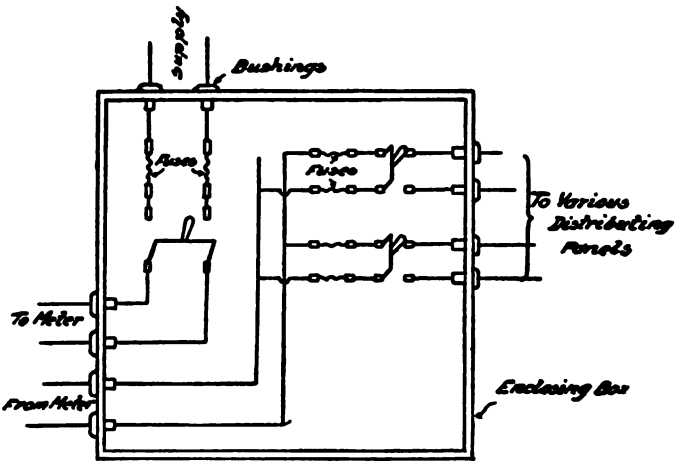


Fig. 11. Showing Location of Meter in Circuit.

with doors and fastenings. They may be mounted on the surface of the wall, or placed in the wall so that door is flush.

The cut-out and service switch should be located in the circuit as near where the wires enter the building as possible.

The meter is installed in circuit just after the service switch (see Fig. 11). Where the installation is not large, the distributing panel may be dispensed with, and circuits run directly from the box shown in Fig. 11.

INSTALLING THE WIRES

There are a number of different methods that may be employed in installing the wires, the selection of the method used depending upon the requirements of the Inspection Department, the initial cost, and the general appearance. The simplest and least expensive method of wiring is to place them upon the surfaces of

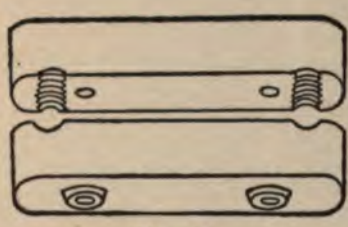


Fig. 12.

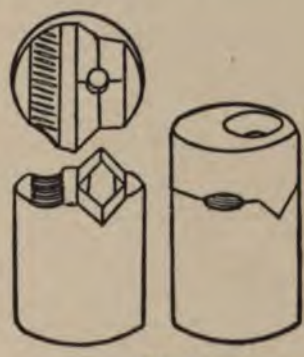


Fig. 13.

Cleats or Knobs Used for Installing Conductors in Open Work.

walls and ceilings, they being held in place by means of porcelain **cleats** or **knobs**, as shown in Figs. 12 and 13. This method is not always permitted by Inspection Departments; moreover, it is unsightly, and wires are exposed to mechanical injury, which is no small consideration. Since the wires are all visible and exposed, this is termed either **open work** or **knob and cleat work.**

A second method, which overcomes to a certain extent the objectionable features of the

method just described, is to place the wires in a wooden or metal **casing** or **moulding,** which is fastened to the walls and ceiling, in plain sight. The wires in this case are more or less accessible for inspection at any time, and have good protection from mechanical injury. There are a number of different types of casings in use. One of the least expensive, and still a very serviceable one, is constructed of wood as shown in Fig. 14. This moulding is made in two parts—a backing and capping—so arranged as to thor-

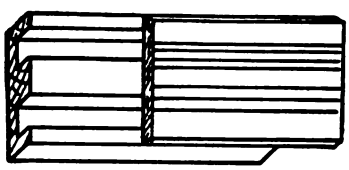


Fig. 14. Wooden Casing or Moulding for Conductors.

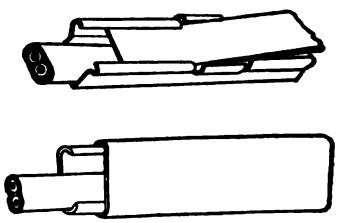
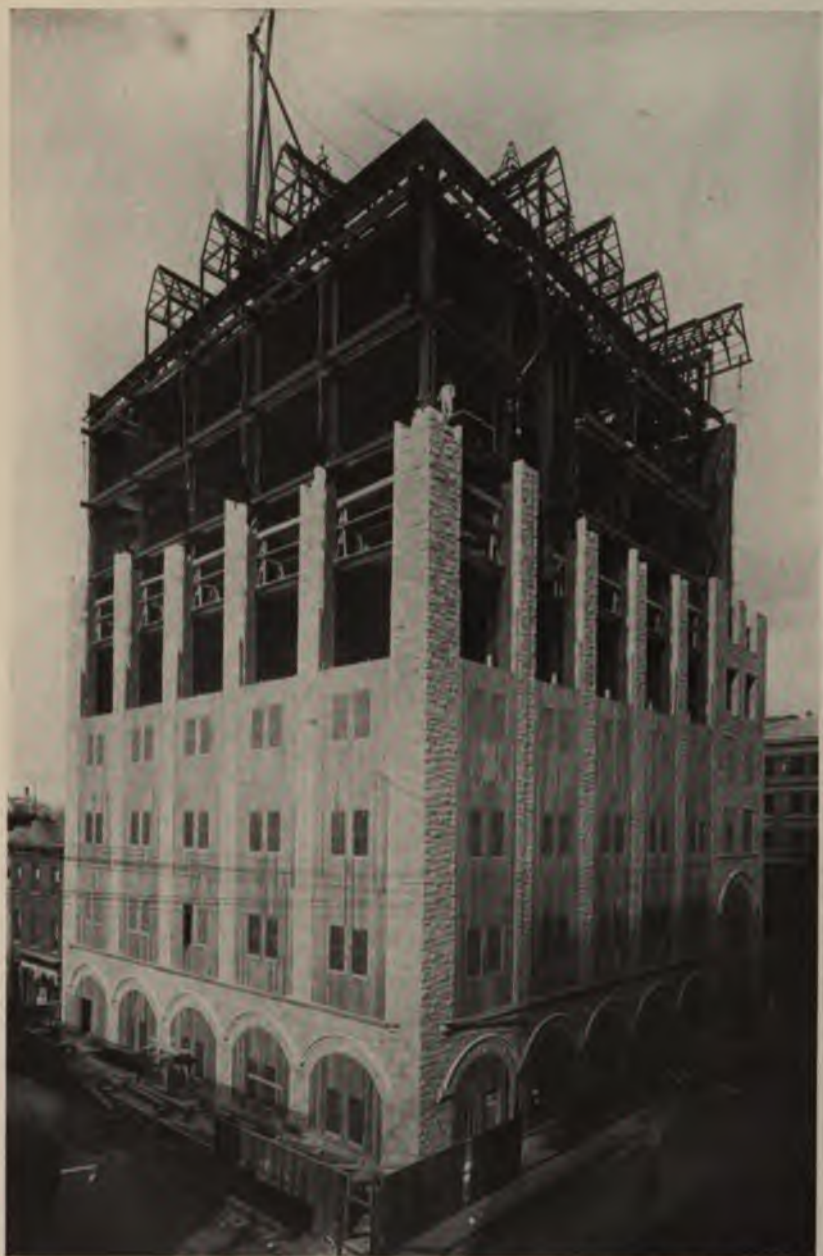


Fig. 15. Types of Metal Casing or Moulding.

oughly encase the wire and afford suitable protection from abrasion. The backing is first screwed to the wall or ceiling; and after the wires are put in place, the capping is nailed on.

There are a number of different forms of metal mouldings on the market at the present time which are more compact than wooden ones and are growing in favor (see Fig. 15).

A means of protecting the wires that is much better than using moulding, is, first, to place them in a **conduit,** which is nothing more than a strong, smooth, water-tight tube having its inner



Courtesy of American Bridge Co.

NORTHWEST BUILDING, NEWARK, N. J., UNDER CONSTRUCTION.

Nine stories high; dimensions, 90 ft. by 106 ft. Weight of steel used, 2,000,000 lbs.

PLATE 38—STEEL CONSTRUCTION.

surface coated with an insulating compound; or, second, to use what is known as **flexible armored cable**, as shown in Fig. 16. These pipes are installed by fastening them in place by means of straps, illustrated in Fig. 17.

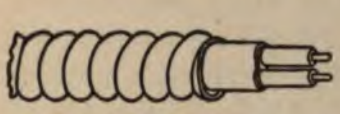


Fig. 16. Flexible Armored Cable.

Fig. 17. Strap for Holding Cable in Place.

All elbows and bends in conduit must be made so that the conduit or insulating lining of

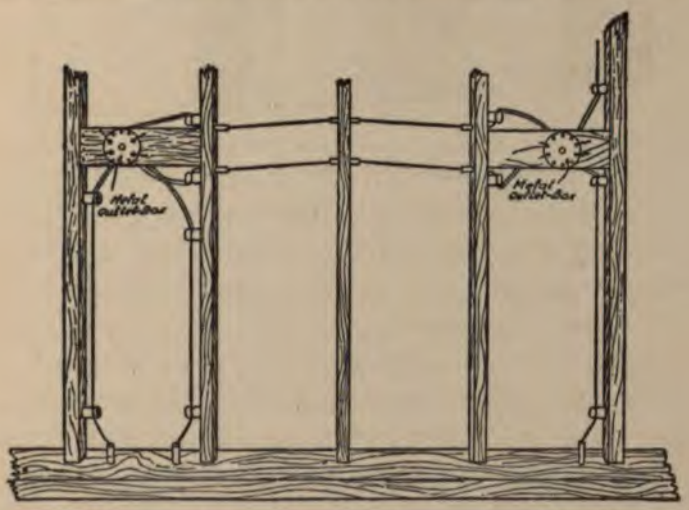


Fig. 18. "Cleat-and-Knob" or "Knob-and-Tube" Method of Installing Wires.
In many of the large cities this method is no longer permitted.

the same will not be injured. The radius of the inner edge of any bend should not be less than 3½ inches, nor should there be more than the

equivalent of four 90-degree bends in a run of conduit from outlet to outlet, neglecting the bend at the entrance to the outlet box. The wires are pulled into the conduit after it is installed, by means of steel tape or fish wire.

Still another method of installing wires is employed, in which they are placed entirely out of sight. In this case, the work is known as **concealed work.** One of the first methods of installing concealed work was to fasten the wires to the ceiling and walls before they were plastered, the wires being entirely covered when the building was complete. This method is a very dangerous one, and is no longer permitted by the Fire Underwriters. The cleat and knob method may be employed where the floors and walls will permit, the wires being placed so that they will not touch anything except their insulating supports, and being passed through insulating tubes in floors, walls, and joists, as shown in Fig. 18. The wires are also encased in a circuit or loom such as is shown in Fig. 19, where they cross or enter outlet-boxes. This is a much safer method than the previous one; but the wires are still exposed to mechanical injury by

Fig. 19. Circuit or Loom Forming a Protective Insulating Covering for Wires where they Cross or Enter Outlet-Boxes.

workmen, such as plumbers and carpenters who are making repairs.

The most satisfactory method of all for installing the wires, is to place the conduit while the building is being constructed, in which case it is placed out of sight and affords the best of protection for the conductors. The conduit is connected to the boxes by means of a lock nut and bushing, which serve as a mechanical connection. The bushing prevents the insulation on the wires from being injured when it is pulled into place.

The importance of using the utmost care in arranging and putting in place electric light wires in a building cannot be too strongly emphasized. The National Board of Fire Underwriters have compiled a set of rules and specifications which govern the installation of electrical apparatus in buildings insured in companies belonging to the association. In addition to this, the Electrical Inspection Departments in some of the larger cities have a set of rules and specifications governing all wiring within the limits of the city. The order of procedure in such a city is to make application to the Electrical Department for a permit to do the wiring. Upon receipt of the permit, the wiring may be installed, and, when completed, it must be inspected by one of the City Inspectors, who determines whether it fulfils all requirements as laid down in the book of Rules and Specifications. When work has been passed upon by the

Inspector, the electrical company is given notice that they may connect to the building and give the desired service. If the city or town does not require such an inspection, it is nevertheless always well to follow the code of the Fire Underwriters, as likelihood of fire is thereby reduced and insurance rates lowered.

INSTALLING ELECTRIC BELLS

The modern building is not considered complete unless it is equipped throughout with **electric bells** to be used for signaling from one position to another. Electric bell wiring, on account of being operated at a low voltage from batteries—which reduces the likelihood of fires caused by short circuits and bad contacts—is not governed by the Inspection Departments of cities, nor by the Fire Underwriters. This fact oftentimes results in installations being poorly made in order to save in first cost, though this practice, as a rule, is more expensive in the end, on account of poor service and cost of repairs. Hence considerable care should be exercised in the selection of materials and the method of installing wires and apparatus.

The various parts composing the bell circuit, no matter what the exact application may be, are: the **bell**, the **push-button** or device for closing or opening the circuit; the **battery**, which supplies electrical energy; and the **conductor** or **wire**, which connects the different parts and forms the circuit.

The **operation of the electric bell** depends upon the following principle. If a current of electricity is passed through a coil of wire surrounding an iron core, the core becomes magnetized and will attract other magnetic substances. Fig. 20 is a diagrammatic representation of the ordinary vibrating bell or buzzer. The circuit through the bell is as follows: from

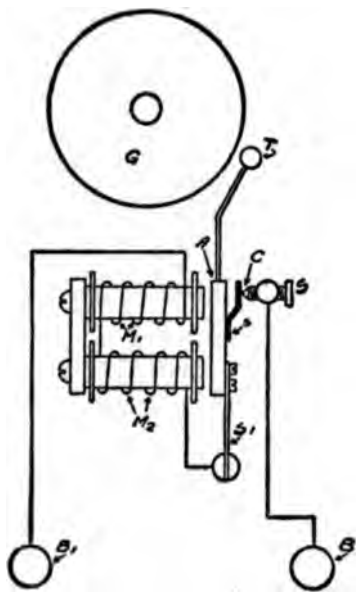


Fig. 20. Vibrating Bell or "Buzzer."

binding post B^1, through windings of coils M_1 and M_2, to spring S^1, through armature **A**, to spring **s**, through contact-point **C**, to adjusting screw **S** to other binding post **B**.

When battery current is sent through coils M_1 and M_2, by closing the circuit of which they

form a part, their cores are magnetized, and they attract the iron armature **A**, which carries the spring **s**. The armature, which is itself supported by the spring S^1, will then move over toward the iron cores until the contact at **C** is broken by the spring **s** leaving the point of the adjusting screw **S**. As soon as the contact is broken, current ceases to flow through the circuit, and the cores in the coils M_1 and M_2 are demagnetized, whereupon the armature, carried by the spring S^1, will fly back again and reestablish the circuit at **C**, again remagnetizing the cores of M_1 and M_2. The process is then repeated, again and again. As a result, the armature is kept flying back and forth, being vibrated as long as there is battery connected to B^1 and **B** and the circuit is closed outside of the bell through the battery.

The armature **A** carries a **tapper** which strikes against the gong **G**. The contact-point **C** should be formed of platinum, as there is a small arc formed when circuit is broken, which eats away other metals very rapidly. All parts of the bell should be well insulated; and the bell should be mounted on a metal back instead of on wood, as the latter is likely to warp and throw the bell out of adjustment. The mechanism of the bell should also be provided with a covering, as this prevents trouble due to dust collecting at the point of contact **C**.

Bells and **buzzers** operating on this principle are manufactured in a number of different

forms, their difference being more mechanical than electrical.

The device employed in closing the circuit is what is termed a **push-button.** This is so constructed that the contacts in the button are held normally open by means of a spring **S** (see Fig. 21). **W** is a circular piece of fiber which supports **S** and **S**^{1}, the two terminals of the circuit, and which is held in place by means of

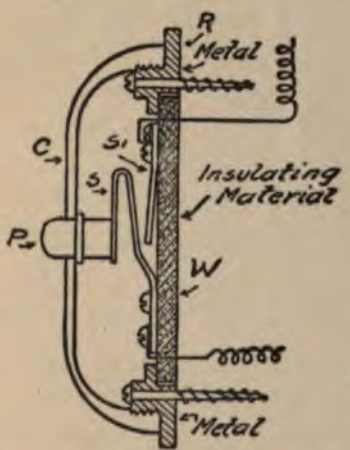


Fig. 21. Section Showing Detail of Construction and Connections of a Push-Button.

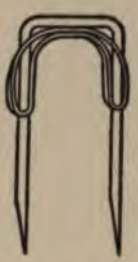


Fig. 22. Staple for Holding Wire in Place.

metal ring **R**, fastened to the wall or other surface by screws. **C** is a metal cap that is screwed onto the ring **R** and holds the push **P** in place. This is a very good form of push-button; but they are manufactured in various other forms and with different finishes.

The **battery** used in bell work should be of the **open-circuit** type, the circuit being normally

open. Two cells are usually sufficient to operate the ordinary bell or buzzer. Good **dry cells** are very satisfactory for this class of work, on account of their cleanliness, as they cannot spill.

The **wire** used should be well insulated, and held in place by staples or cleats, making sure that there are no short circuits due to staples cutting the insulation. A good form of staple to use is shown in Fig. 22. It is found to be quite a great advantage to have the insulation of the wires in different colors, so that they may be easily indentified without testing, thus saving time when installing. The sizes of wire commonly used are Nos. 18, 20, and 22, B. & S. (Brown & Sharpe) gauge. Wires smaller than these are too weak from a mechanical standpoint.

Figs. 23 to 30 show how bells may be wired

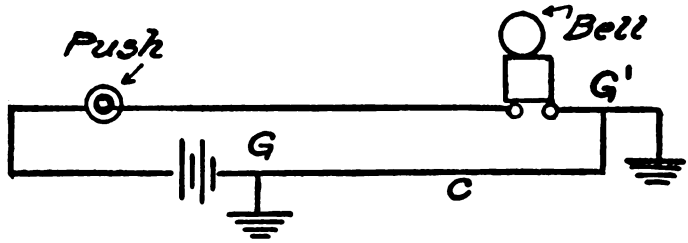


Fig. 23. Simplest Form of Bell Circuit.

to accomplish different results. Fig. 23 shows the simplest circuit we can have. There can be a saving in wire by using what is termed a **ground**, the current being returned by connecting to ground at **G** and G^1, thus dispensing with

the conductor **C**. A ground connection may be made by connecting to gas or water pipe. The number of pushes controlling this bell can be increased as shown in Fig. 24.

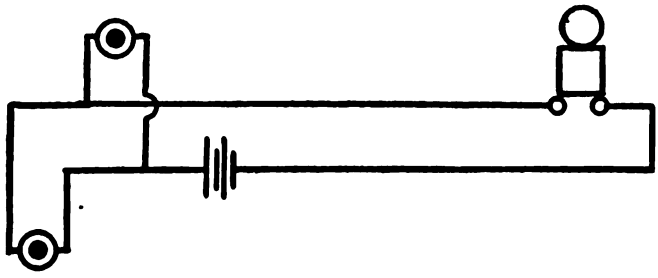


Fig. 24. Method of Increasing Number of Push-Buttons Controlling a Bell.

The number of bells controlled by a push-button can be increased as shown in Figs. 25 and 26. The bells are connected **in multiple** in

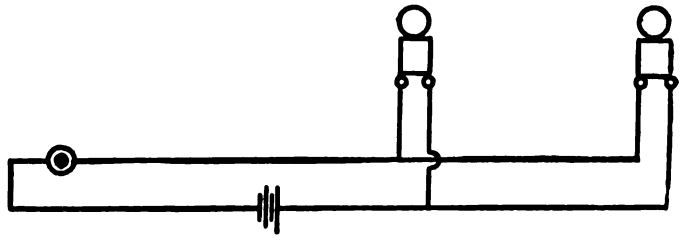


Fig. 25. Method of Increasing Number of Bells Controlled by a Single Push-Button.
Bells connected in multiple, each taking current independently of the others.

Fig. 25, and each takes current independently of the others. In Fig. 26, they are connected **in series**; only one bell in this case is of the vibrating type, it serving to make and break the circuit through the others.

Annunciators. Quite often it is desirable to

have a bell ring from a number of different places, such as different rooms of a house, and a means must be provided for indicating the

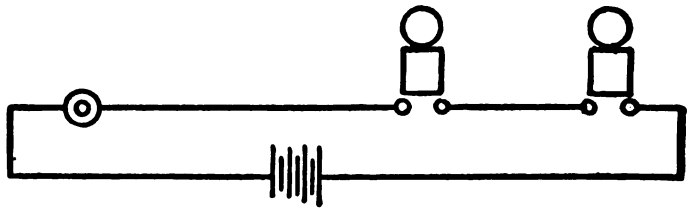


Fig. 26. Several Bells Controlled by a Single Push-Button. Bells connected in series, one vibrating bell making and breaking circuit through other bells.

push from which the bell was rung. A device called an **annunciator** fulfils this requirement.

Fig. 27 shows the circuit of a simple annunciator, there being only four drops shown.

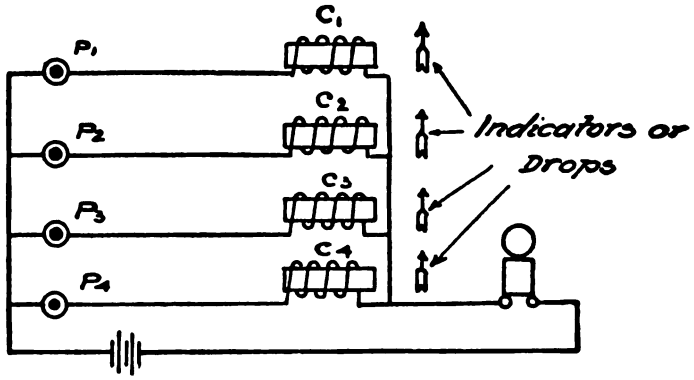


Fig. 27. Circuit of a Simple Annunciator.

When any push is operated, there is sent through a coil in the annunciator a current corresponding to that push which causes the indicator to drop, thus giving an indication of where the call was made from.

The indicators are restored to their original position by means of a lever at the bottom of the box, or may be restored electrically by pressing a push-button for this purpose.

Fig. 28 shows an annunciator suitable for residence work.

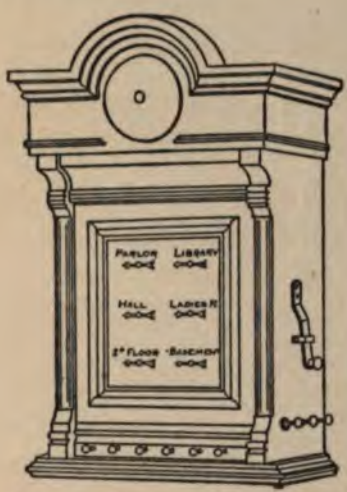


Fig. 28. Annunciator for a Residence.

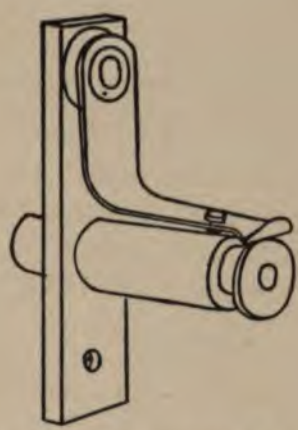


Fig. 29. Contact for Open-Circuit Burglar Alarm.

Burglar Alarms. The electric bell may be used as a **burglar alarm,** by having the circuit so arranged that a bell will ring when anything is disturbed—as, for example, by opening a door, by raising a window, by breaking a glass, etc.

The simplest circuit is practically the same as those just described, except that contacts are placed on doors, windows, etc., instead of using the push (see Fig. 29), and are so arranged

that they will complete the circuit when the doors are opened or the windows raised. This is what is termed an **open-circuit alarm,** as the circuit is normally open.

Another type, known as the **closed-circuit,** in which the circuit is normally closed, is used in the following manner: A closed-circuit battery is connected in series with a relay, and, say, a strip of gold foil across a plate-glass window. This relay is normally energized; but if the glass is broken and the circuit thereby opened,

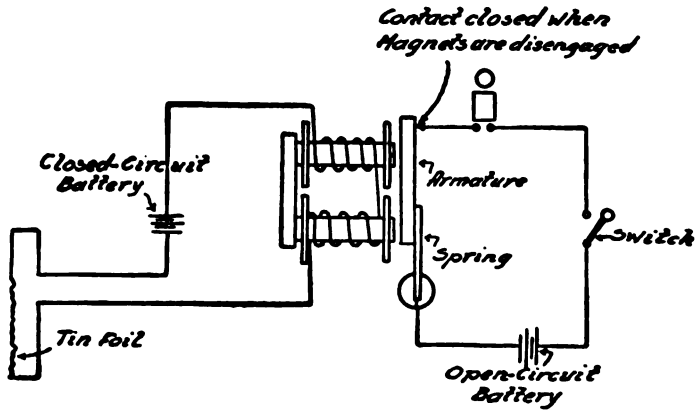
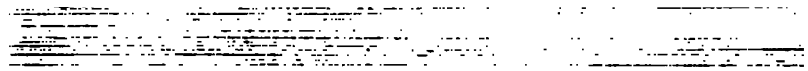


Fig. 30. Diagrammatic Representation of Closed-Circuit Burglar Alarm, Showing Connections.

the relay becomes de-energized, and its armature drops back, completing a local circuit containing a vibrating bell (see Fig. 30).

Electric Fire Alarm. Another and important application of the electric bell is that of the **electric fire alarm.** The operation of the device controlling the circuit may depend upon

the expansion of a certain metal, due to temperature rise, completing the circuit. Or a portion of the circuit may be composed of a metal that will melt at a rather low temperature and open the circuit in case of fire. This first would be the **open-circuit** type; while the second would be the **closed-circuit** type.



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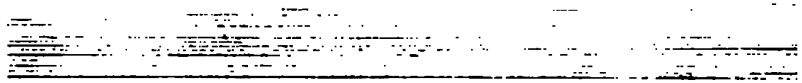

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